DESIGN OF A THREE-STORY REINFORCED CONCRETE COLD STORAGE BUILDING

BY

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ARMOUR INSTITUTE OF TECHNOLOGY

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THE STRUCTURAL DESIGN
OF A
THREE-STORY REINFORCED CONCRETE COLD STORAGE BUILDING

A THESIS

PRESENTED BY
GAGE REX WAMSLEY
TO THE
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MAY 31, 1920

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--- T H E S I S ---

DESIGN OF A THREE-STORY REINFORCED CONCRETE COLD STORAGE BUILDING.

O B J E C T .

This thesis embodies the required calculations and drawings for the complete structural design of a reinforced concrete cold storage building. The problen involves the determination of the stresses in the slabs, beams, columns and footings; the proper disposition of the reinforcement for strength and stability; a compilation of typical details to clearly illustrate the construction; and an estimate of the cost.

In the computations of all structural members and in the general design, the Chicago Building Code will govern.

The subject matter naturally divides itself into these subheads which will be treated in the following order;

A. Descriptive and dimensional data.
B. Structural design.
C. General architectural drawings.
D. Reinforcing plans.
E. Constructional details.
F. Cost investigation.

A tabulation of the allowed unit stresses and a legend of symbols used in formulas are given at the end of the thesis.

DESCRIPTIVE AND DIMENSIONAL DATA.

DIMENSIONS. The location of the building on the site and its relation to the property lines are shown on the survey plat, Sheet #1. The structure itself is square but the property is in the form of a trapezoid with only three sides at right angles to each other, consequently the flat-iron shaped corner may be utilized for the office which is to be one story in height. The main building however is three stories in height above the basement.

The first story embodies the wagon platform, the shipping platform, the office, a city trade cooler for domestic sales, beef freezer and beef cooler, a storage cooler room for packed meats, and an air lock to platform.

On the first floor plan are given the general dimensions; length 90' 0" divided into six bays of 15' 0" each; width 68' 0" divided into four bayes of 17' 0" each; the office projects 17' 0" farther and includes two bays 15' 0" each.

The height from first to second floors is 11' 8"; from second to third, 11' 8"; from third to low point of roof, 11' 8" and to high point 2'0" more.

The second floor is to be devoted to storage space for dried fruits, canned meats, and the third floor to butter eggs, lard and dried meats.

The basement will be used for general storage and for

DESCRIPTIVE AND DIMENSIONAL DATA.

keeping hides. It will also accommodate all necessary leaders for piping from power house to cold storage building.

DESCRIPTION OF STRUCTURAL DESIGN. The building is to be practically fireproof by using concrete throughout, with steel window frames and rolling steel shipping platform doors. The partitions will be either tile or cork or both in combination; but in any case plastered on the inside.

The elevator will run from basement to penthouse on roof, and there are to be two separate stairways from top to bottom of building to comply with the City Ordinance.

For insulation, cork board will be used, the building to be completely enveloped or enclosed in it to make the insulating effect as perfect as possible. This may be done by using a layer of cork along the inside walls entirely separating them from the interior portion by constructing a framework to support the floors independently of the walls, as indicated by the drawings.

A layer of cork will run under the first floor and over the roof slab making a complete cork envelope for the interior.

The exterior walls will consist of brick panels supported from a system of concrete columns and spandrel beams. The brick will be extended around the outside of columns and beams to make the exterior uniform in architectural treatment.

Windows will be provided in the stair well and office only, the cold storage portions to be lighted by electricity, and

DESCRIPTIVE AND DIMENSIONAL DATA.

ventilated by a system of ducts and fans.

The roof will slope in two directions away from the ridge at the center of the building and drain spouts are to be provided at intervals along the coping wall to carry off the roof drainage. A heavy layer of tar and gravel roofing will be put directly upon the top layer of cork insulation to make a thoroughly waterproof job and to assist the cork in keeping out the heat.

-- STRUCTURAL DESIGN --

PRELIMINARY COMPUTATIONS:-

In the design of reinforced concrete, the first considerations involve the selection of the unit working stresses of the materials, the quality of steel and grade of concrete to be used, the relation of their moduli of elasticity, and the values of constants which occur in the calculations.

The regulations of the Chicago Building Code, which are to govern in this case, specify the following allowable stresses for 1:2:4 Portland cement concrete, the standard mixture best suited for the construction of slabs, beams and columns.

Bending, extreme fibre, f = 700 lbs. per square inch.

Direct compression, c = 400 lbs. per square inch.

Shear in diagonal tension, v = 40 lbs. per square inch.

Bond between concrete and plain round bars (for slabs), u = 50 lbs, per square inch; between concrete and deformed round bars (for beams and columns), w = 100 lbs. per square inch.

High carbon steel will be chosen, having an elastic limit of not less than 55000 lbs. per square inch, thus permitting the following values:-

Tension, s = 18000 lbs. per square inch.

-- STRUCTRUAL DESIGN -- (Cont'd)

Shearing tension (for stirrups), y = 12000 lbs. per square inch.

Ratio of modulus of elasticity of steel to that of concrete, n = 15.

Compression, when used in columns, m = n x f = 15 x 700 = 10500 lbs. per square inch.

Hooked bars, having a semi-circular hook with radius three times diameter of bar shall be considered capable of developing their full tensile strength.

Steel stirrups shall take the stresses due to vertical shear in excess of 40 lbs. per square inch. for the concrete, but the combined shear must not exceed 133 lbs per square inch.

When steel is used in the compression side of beams, the rods shall be tied by stirrups at intervals of 12 diameters of bar.

For protection against fire the steel must not be nearer the surface than 1 1/2 inches for beams and columns; 1/2 inch for slabs. To secure proper bond the spacing between centers of bars shall be 2 1/2 diameters; and not less than one inch between layers of bars in beams or girders.

-- STRUCTRUAL DESIGN -- (Cont'd)

Formulas, derived in Turneaure and Maurer's "Principles of Reinforced Concrete", recognized in standard practice, will be used in all computations. The quantities p, k, j, R etc., required in slab and beam calculations are therefore obtained as follows:-

The economical steel ratio, producing the maximum allowable stresses to exist in both steel and concrete at the same time equals,

$$p = \frac{1}{2} \cdot \frac{1}{s/f\ (s/nf + 1)} = \frac{.5}{18000/700\ (18000/15 \times 700 + 1)}$$

$$= \underline{.0072} = .72\%$$

Ratio of depth of neutral axis to depth of steel, equals,

$$K = \sqrt{2pn + (pn)^2} - pn = \sqrt{(2 \times .0072 \times 15) + (.0072 \times 15)^2} - (.0072 \times 15)$$

$$= .476 - .108 = \underline{.368}$$

Ratio of arm of resisting couple to depth of steel equals,

$$j = 1 - k/3 = 1 - .368/3 = 1 - .123 = \underline{.877}$$

When the percentage of reinforcement equals the economical steel ratio, the resisting moments of the steel and concrete are the same, and are represented by Rbd^2. Then,

For the steel $R = spj = 18000 \times .0072 \times .877 = \underline{113}$

For the Concrete $R = .5\ fkj = .5 \times 700 \times .368 \times .877 = \underline{113}$

--STRUCTURAL DESIGN--

REFERENCE TABLE FOR BEAM CALCULATIONS.

For determining the compressive stress in the concrete at the top of rectangular beam, or at the support of a continuous T-beam where the lower half is in compression, the formula,

$f = \frac{M}{.5\ k\ j\ b\ d^2}$ comes into use. The quantities M, b and d are known but the quantity .5k j varies with the steel ratio p, a variable, and n which in this case is taken at 15.

$k = \sqrt{p^2\ n^2 + 2\ p\ n} - p\ n$ and $j = 1 - \frac{k}{3}$. It is therefore convenient to tabulate k, j and .5 k j, the latter designated by K, for various values of p for use in the formula given above. This tabulation is given herewith and will be referred to as Table #3.

TABLE #3.

p	k	j	K
.005	.319	.893	.143
.007	.365	.878	.160
.009	.402	.866	.174
.010	.412	.861	.180
.011	.433	.856	.185
.015	.483	.839	.202
.018	.513	.829	.213
.020	.531	.823	.218

-- STRUCTURAL DESIGN --

FIRST FLOOR SLAB.

The live load per square foot on the first floor will be taken at 200# per square foot to allow for heavy storage and for trucking. A slab slightly thicker than required by the computations is desirable to offset the wear on the floor. The slab will be re-inforced in two directions and as the inverse ratio of the third powers of the two spans gives a coefficient of .59 as for the roof, the loading w in both directions will be as follows:-

Live load per square foot	= 200 #
Dead load, slab 81#, Cork 4#	= 85
Total	= 285 #

w = load taken by transverse rods = .59 x 285 = 168#

w' = " " " longitudinal " = .41 x 285 = 117#

For the typical interior continuous spans,

$M = 168 \times 15 \times 15 \times 1 = 37800$ in. lbs.

$M' = 117 \times 17 \times 17 \times 1 = 33900$ " "

$$d = \sqrt{\frac{M}{R\,b}} = \sqrt{\frac{37800}{113 \times 12}} = 5.3 \text{ "}$$

$$d' = \sqrt{\frac{M'}{R\,b}} = \sqrt{\frac{33900}{113 \times 12}} = 5.0 \text{ "}$$

Fireproofing	= 0.75	
d (bottom layer)	= 5.3	
Slab thickness	= 5.85	(Transverse)

-- STRUCTURAL DESIGN --

Fireproofing = .75
d' (top layer) = 5.0
Space for lower rods = 0.5
Slab thickness = 6.25

Therefore the slab should be 6 1/2 inches, this being determined by the depth needed for the upper layer of steel.

As explained for the roof slab, the bars should be bent up at the quarter points and carried to the third points in the adjacent spans, half the bars being bent and half straight.

The second floor slab will also be $6\frac{1}{2}$" the same as the first because the live load is also 200 lbs. per square foot as stated in the column computations. Therefore the slab bars will be bent the same as the first floor slab bars.

In computing the slab bars, consider the lower layer as containing the bars extending the short way since they will receive the larger part of the load; then the upper layer will consist of the bars running the long way. First figure the lower layer:-

M = 37800 "# d = 5.75"

$$\text{Steel area} = \frac{37800}{18000 \times .877 \times 5.75} = .416$$

This is equivalent to $\frac{1}{2}$" round rods at $5\frac{1}{2}$" centers for the middle half of the slab; for the outer quarters space the rods farther apart, namely 10" centers.

For the spacing in the upper layer:-

M' = 33900"# d' = 5.25

-- STRUCTURAL DESIGN --

$$A = \frac{33900}{18000 \times .877 \times 5.25} = .373$$

This is equivalent to $\frac{1}{2}$" round rods spaced $6\frac{1}{2}$" centers for the middle half of the slab; for the outer quarters space $\frac{1}{2}$" round rods at 10" centers.

The rods lying in the outer quarters will be straight, extending from center to center of supports only; while the rods lying in the middle half will be bent up at the quarter point, the alternate rods in each span running over the supports and to the third points in the adjacent spans. By this arrangement there will be the same amount of steel over the supports as at the middle of the span, conforming to the assumption that the bending moments for the usual conditions of loading are approximately the same at the support as at the center of the slab. Where the slab is continuous over one support only as at the wall, the rods will be run straight along the bottom of the slab into the wall or spandrel beams on the supposition that no appreciable negative moment exists there and that only a negligable amount can be resisted by the beam as a torsional stress.

-- STRUCTURAL DESIGN --

SECOND FLOOR BEAMS. (1st fl. bms. similar)

The loading on the second floor per square foot is ... 200 #

Dead load including cork insulation and concrete slab. 85

285

Multiplying the coefficient of .59 by 285 gives the transverse

load = ... 168 #

Then the longitudinal load = ... 117 #

For the 15-foot span typical interior beams the triangular load

from the slab = 15 x 17 x 117 = ... 30000

Weight of beams reduced to equivalent triangular load = ... 1700

Beam load W = 31700 #

M = 1/6 Wl = 31700 x 180 x .167 x .8 = 765000"#

Use a 12" x 22" size; d = 20"

$$A = \frac{765000}{.877 \times 18000 \times 20} = 2.4 \text{ sq. ins.}$$

This requires 4 rods 7/8" diameter.

$$V = 15800'' \qquad v = \frac{15800}{12 \times 22 \times .877} = 69\#/ \text{ sq. in.}$$

$$\text{The steel ratio} = \frac{2.4}{12 \times 20} = .01 = p$$

From Table #3, the value of K = .18 and the compressive stress at the

$$\text{column} = \frac{765000}{12 \times 20 \times 20 \times .18} = 880 \text{ \# per sq.-inch.}$$

This is not excessive because the allowable compressive stress of 700 # may be increased about 15% at the support.

In spacing stirrups, the concrete is to be figured for its

-- STRUCTURAL DESIGN --

share of the shearing stress at 40# per sq. inch.

Then total shear =	15800
Concrete takes 12 x 20 x 40 x .877 =	8400
For stirrups =	7400

$$\text{Spacing} = \frac{.11 \times 2 \times 12000 \times .877 \times d}{V} = \frac{2320\ d}{V}$$

$$\text{Spacing at ends} = \frac{2320 \times 20}{7400} = 6''$$

Maximum spacing = .75 x 20 = 15"

Therefore space 3 stirrups at 6", 3 at 8" and 3 at 10" from each end.

For the 17 foot span typical interior beams the triangular load from the slab is 15 x 17 x 168 = 43000#

Weight of beam as equivalent triangular load	= 3000#
Beam load W	= 46000#

M = 1/6 Wl = 46000 x 204 x .167 x .8 = 1,260,000 "#

Use 14" x 30" size; d = 28"

$$A = \frac{1260000}{18000 \times .877 \times 28} = 2.85 \text{ sq. in., requiring 4 rods 1'' diameter.}$$

$$\text{The steel ratio} = \frac{2.85}{14 \times 28} = .007 = p$$

From Table #3 the value of K = .16

$$\text{Compressive stress at support} = \frac{M}{Kbd^2} = \frac{23000}{.16 \times 14 \times 28 \times .877} = 720\#$$

per square inch, which is amply safe.

$$v = \frac{23000}{14 \times 28 \times .877} = 68\ \#\ \text{per Sq. inc.}$$

-- STRUCTURAL DESIGN --

Spacing of stirrups at ends $= \frac{2320 \times 28}{9300} = 7"$. The maximum spacing is $.75 \times 28 = 21"$, therefore space 4 stirrups at 6", 3 at 8" and 3 at 12". Bend up two beam bars at the quarter points and extend to the third point into next beams.

-- STRUCTURAL DESIGN --

ROOF SLAB. - TYPICAL BAY.

The bending moments in slabs as required by the Chicago Code are:

$M = 1/12\ w\ l^2$, for continuous intermediate spans.
$M = 1/10\ w\ l^2$, " " end "
$M = 1/8\ w\ l^2$, " " simple "

The length of span for continuous slabs is from center to center of suports; and the same for non-continuous slabs, except that it need not exceed the slear span, plus the thickness of slab. For rectangular panels where the ratio of the spans does not exceed 1.25 the slab may be reinforced in both directions, the load being distributed in the two directions inversely as the cubes of the spans.

The typical panels measure 17 x 15, having a ratio of 1.13 therefore less than 1.25 and may be reinforced both ways. The proportion of the load carried by the transverse reinforcing will be $\dfrac{17^3}{17^3 + 15^3}$ = 4913/8288 = 59% the short way; and 41% for the long way.

The load per square foot includes:

Live load = 25 #
Dead load
Roofing = 10
4 1/2" Concrete Slab= 56
4" Cork insulation = 4
Total 95 #

-- STRUCTURAL DESIGN --

w (transverse) = .59 x 95 = 56 #

w' (longitudinal) = .41 x 95 = 39 #

For typical roof slab panel, 17' x 15', intermediate continuous spans,

$$M = 1/12\ w\ l^2 = 56 \times 225 \times 1 = 12600 \text{ inch-lbs.}$$

$$M' = \text{"} \quad \text{"} = 39 \times 289 \times 1 = 11300 \quad \text{"} \quad \text{"}$$

$$d = \sqrt{\frac{M}{R\ \ b}} = \sqrt{\frac{12600}{113 \times 12}} = 3.05\text{"}$$

$$d' = \sqrt{\frac{M}{R\ \ b}} = \sqrt{\frac{11300}{113 \times 12}} = 2.92\text{"}$$

Allowing 1/2" fireproofing, and 1/4" to center of 1/2" round rods, makes the distance from bottom of slab to center of steel 3/4". For the lower layer of bars, d = 3.05 and adding 3/4" gives 3.8 for the thickness; for the upper layer, d = 2.92", and adding 3/4" plus 1/2" for bottom bars, gives 4.17" for the thickness; or pay 4 1/2", choosing the nearest half inch. Then the actual depth becomes 3 3/4" for the lower layer and 3 1/4" for the upper layer. Using these values, calculate the area of steel required per foot of slab.

$$A = \frac{M}{j\ d\ s} = \frac{12600}{.877 \times 3.75 \times 18000} = .21 \text{ sq. in.}$$

$$A' = \frac{M'}{j'\ d'\ s'} = \frac{11300}{.877 \times 3.25 \times 18000} = .198 \text{ sq. in.}$$

Round rods 1/2" diameter have a cross-sectional area of .1963 sq. in. To figure the spacing of the rods multiply the cross-section area by 12 and divide by A.

-- STRUCTURAL DESIGN --

Spacing for lower rods = 12 x .1963 = 2.36, dividing by .21 = 11" centers.

Spacing for the upper rods = 2.36 divided by .198 = 12 centers. The spacing in both directions will be made 9 inches however because it is not good practice to space rods farther apart than twice the thickness of the slab (in this case 2 x 4 1/2 = 9") owing to the tendency of the concrete to crack and break between widely spaced rods, especially under jarring loads.

It would be feasible to use 3/8" rods at closer spacing but they are more expensive to handle and the consequent small saving in amount of steel would be offset by the labor expense.

The proper bending of the rods is based upon the location of the points of maximum positive and negative bending moments under any possible conditions of loading which will produce the greatest stresses. In a series of continuous spans with every alternate one loaded the largest positive moment will be produced, closely approximating $1/12 \; w \; l^2$. This is the same value used for the negative moment which reaches its peak when the spans adjacent to the support under question are loaded and also every alternate span thereafter. Under full dead and live load the negative would exceed the positive bending moment yet the maximum values must be provided for in safe design. Under varying conditions the change from positive moment at the center to negative at the support occurs at or near the quarter points, and it is here that

-- STRUCTURAL DESIGN --

the rods should be bent up at an angle of 45 degrees, conforming very nearly to the direction of the diagonal shear forces existing there. The bent up rods should run over the support and into the next span to the third point to take any negative moment which may come about through special conditions of loading above described. The details of the bending are given on the steel lists in the latter part of the thesis.

The slab in the middle half of the bay is subjected to the greatest strain and the part around the edges, being near the supports, possesses additional strength so that the rods should be spaced the minimum distance apart in the middle half and farther apart in the outside quarters; also the rods here may be straight, extending only to the supports. This omission of top steel near the supports permits easy handling and placing of the steel, and at the same time conforms to good practice.

ROOF BEAMS - TYPICAL INTERIOR 15' SPAN.

Where slabs are supported on four sides the beams do not carry a uniform load but more nearly a parabolic or triangular shaped load, the latter usually being figured as it is slightly on the side of safety, the bending moment equalling 1/6 W 1 modified. The 15' beam receives the load carried to it from both sides by the longitudinal slab reinforcement, amounting to 37# per sq. ft. over an area 17 x 15 feet, totaling 9435 lbs. triangular

--STRUCTURAL DESIGN--

load. The weight of the beam itself at 200# per ft. is 3000#. This might be figured separately as M = 1/8 W 1 but may be combined with the 9435 lbs. by using only 3/4 of it, since that is the ratio of 1/8 W 1 to 1/6 W 1. Because of the continuity 1/6 W 1 should be reduced until equivalent to the coefficient 1/12 W 1 as compared to 1/8 W 1. This reduction is 33 1/3% but is not taken less than 30%.

Total triangular load 9435 + 2250= 11685 #

M = .7 x 1/6 Wl = .7 x 2 x 11685 x 15 = 245000"#.

V = .5 x 11685 = 5850 #.

Four bars will be used so that two may be bent up and carried into the next span to take care of negative moment at the support. With four bars in one layer spaced 2 1/2 diameters apart to insure proper bond, a width of at least ten inches would be needed, allowing for 1 1/2" concrete protection for the steel on each side. For a tentative size a 10" x 16" beam will be selected. The depth to steel will be 14". Then :- $A = \frac{245000}{18000 \times .877 \times 14} = 1.1$ sq. in.

This is equivalent to 4 - 5/8" round bars, whose total area is 4 x .3 = 1.2 Sq. in.

$p = \frac{1.2}{10 \times 14} = .0086$ At the support the bottom of the beam is in compression and there being no T flange, the formulas for rectangular beams will apply. When p = .0086 then k j = .169 (See Table #3)

$$f = \frac{M}{2\ b\ d^2 k\ j} = \frac{245000}{,0 \times 14 \times 14 \times .169} = 740 \text{ \# per sq. in.}$$

-- STRUCTURAL DESIGN --

which is sufficient, 800 # being allowed adjacent to the support.

$$v = \frac{V}{b\ d\ j} = \frac{5850}{10 \times 14 \times .877} = 48 \text{ \# per sq in.}$$

$b \text{ (required)} = \frac{M}{R\ d^2}$ Now, when $t/d = \frac{4.5}{14} = .32$, then

R = 108 from Table 4.

$b = \frac{245000}{108 \times 14 \times 14} = 11\ 1/2"$ This is not too great since it may be as large as 1/3 the span between beams.

It will be noted that the size of the beam in this case is governed by the negative bending moment ever the support and not by shear or width of T-flange. Of course compression steel could be used but the beam is as shallow as good practice would permit for such a long span and furthermore it is preferable not to use compression steel over the supports except for the end spans. Where compression steel is used, it should be limited to 1% since if an excessive amount of steel is used the formulas may fail to represent the true relation between the concrete and steel stresses.

The end span beams are designed similarly but $M = 1/10\ w\ l^2$ instead of $1/12\ w\ l^2$, and 1/6 W l should be reduced 20%

$$M = .8 \times 2 \times 11685 \times 14.5 = 270000"\#$$

$$A = \frac{M}{s\ j\ d} = \frac{270000}{18000 \times .877 \times 14} = 1.23 \text{ Sq. in.}$$

Use 2-3/4" and 2-5/8" round bars, total area 1.48 sq. in. The negative moment at the support will exceed the positive moment

-- STRUCTURAL DESIGN --

at the center, therefore more steel will be needed at the support so the two larger 3/4" bars should be bent up and the two 5/8" bars run straight along the bottom of the beam.

For stirrups 3/8" round rods will be used to take the shear in excess of that carried by the concrete at 40# per sq. in. When steel is used in shear action the allowable stress is 12000# per sq. in.

Let V = total shear in beam.

" V' = shear taken by concrete

" V'' = " " " stirrups.

Then $V'' = V - V'$ and the stirrups should be designed for this difference, according to the formula, $\text{SPACING} = \frac{s' \times j\,d \times A}{V''}$,

where A is the cross section of the two prongs of the stirrup.

Substituting values:-

$\text{Spac.} = 12000 \times .877 \times .22 \times \frac{d}{V''}$, since A for 2 prongs

3/8" round $= 2 \times .11 = .22$

$\text{Spac.} = 2320 \times \frac{d}{V}$

At the end of the beam $V = 5850$ #

$10 \times 14 \times 40 \times .877 = V' = \underline{4900}$

$V'' = 950$ #

Spac. at ends $= 2320 \times \frac{14}{950} = 34"$ but maximum spacing should not exceed 3/4 the depth of beam which is 12". Therefore use arbitrary spacing of 3 at 8" and 3 at 10"

-- STRUCTURAL DESIGN --

ROOF BEAMS - TYPICAL INTERIOR - 17'0" SPAN.

The floor load carried to the 17 ft. span beam by the transverse reinforcement is 56 x 17 x 15 = 14300#, considered as a triangular load. The weight of beam taken as 200# per ft. = 3400#; to combine with the triangular load add 3/4 of 3400 = 2600# (which will produce the same bending moment); total = 16900# = W.

M = .7 x 1/6 Wl = .7 x 12 x 16900 x 17 = 402500"#

V = .5 x 16900 = 8450 #

The size of the beam selected involves consideration of the shearing resistance, limitation in diameter and number of reinforcing bars, space for steel, and value of negative bending moment at the support.

Four bars is a convenient number for continuous beams, allowing half of steel to be bent over the supports and into the next beam without interference. A width of 12" would accomodate four bars 1" diameter or less, assuming a distance of 2 1/2 diameters between bars, and 1 1/2" from side surface of beam to the steel. A depth of 16" will fulfill the condition that the stress in the concrete at the support shall not be excessive. The depth to steel d = 14".

It should be noted that where a continuous beam passes over the supports, the top is in tension and the cottom in compression with no T flange to assist so that the resistance of a rectangular,

-- STRUCTURAL DESIGN --

not a T beam must be figured.

$$f = \frac{M}{.5\ b\ d^2\ k\ j} = \frac{402500}{12 \times 14 \times 14 \times .195} = 885 \text{ \# per sq. in.}$$

The quantity .193 is taken from Table #3 and is the value when

$p = .0125$

$$A = \frac{M}{s \times j\ d} = \frac{402500}{18000 \times .877 \times 14} = 1.8 \text{ sq. in.}$$

Use 2 round bars 3/4" and 2-3/4". Area 2.08 sq. in.

$$v = \frac{V}{b\ j\ d} = \frac{8450}{12 \times .877 \times 14} = 58\ \#$$

At the support the concrete stress must be reduced by compression reinforcement. The stress must be reduced to 800#. The difference is 885 - 800 = 85# per sq. in. By using steel in the compression side which is the lower portion of the T-beam at the support, the reinforcement will relieve the concrete of some of its stress. This is done by letting the bottom rods run through or by using extra short rods over the supports. The former method will be used and one of the two bottom bars in each beam will be extended into the adjacent spans. By diagram #4 it will be seen that when a reduction of 25% is needed, only 1% of compressive steel is required and in this case where the reduction is small 1 bar as a minimum will be run through.

The bending of the bars in the continuous beams is governed by the same considerations as in the case of slabs so that the bars will be bent up at the quarter points and extended over the top of the supports to the third points in the adjoining spans.

-- STRUCTURAL DESIGN --

COLUMNS.

A convenient diagram for reference in designing concrete columms is that shown as Plate #5. Its construction consists in laying off on the x axis of coordinates the percentage of vertical sheel, p, which ranges according to the Chicago Code from .005 to .03; and laying off on the y axis the load on the column. Then selecting standard size such as 12 x 12 or 16 x 16; the graphs are made for each. These graphs must be straight lines because the formula for columms is in the first degree. $P = A\,c\,(1 + p\,(n - 1))$ For example for a 14" x 14" column when $p = .005$ then

$$P = 121 \times 400\ (1 + .005 \times 14) = 48400 \times 1.07$$
$$= 51800 \text{ lbs.}$$

and when $p = .03$ then,

$$P = 48400 \times 1.42 = 68600 \text{ lbs.}$$

To determine a column to hold 55000 lbs. for instance, refer to Table #5 and trace horizontally from 55000 to the point where it intersects the graph for 14 x 14 column, then vertically downward to .0098.

-- STRUCTURAL DESIGN --

COLUMNS-TYPICAL INTERIOR.

The floor area carried by each interior column is 17 x 15 = 255 sq. feet in each shorty. To this must be added the weight of the adjacent beams and the column itself. The live loads per square foot are, for the several floors, 200# on first and second, 150# on third, and 25# on roof. The dead loads are, first floor 85#, second 85#, third 80#, roof 70#; see floor slab design sheets. The average weight of floor beams framing into the column is 6500#.

These values tabulated are as follows:

LOADING ON TYPICAL INTERIOR COLUMNS.

					TOTAL
3rd story	255 x 95	=	24200		
	Beams	=	6500		
	Column	=	1600	32300	32300
2nd Story	255 x 210	=	53500		
	Beams	=	6500		
	Column	=	2000	62000	94300
1st Story	255 x 245	=	62200		
	Beams	=	6500		
	Column	=	3400	72100	166400
Bsmt. Story	255 x 235	=	60000		
	Beams	=	6500		
	Column	=	4000	70500	236900

-- STRUCTURAL DESIGN --

In the above tabulation 100% of the roof live load is taken; 85% of the 3rd floor live load; 80% of the 2nd floor live load; and 75% of the 1st floor live load; in all cases using 100% of the dead load according to the Chicago Code.

For the third story column a 12" x 12" size may be used with a minimum amount of steel, p = .005 as will be seen by referring to Table 5. No smaller column would do because the story height is 11' 8" and the ratio of height to least side must not exceed 12. The core area is 9 x 9 = 81 = 405 sq. in. but the total area of steel cannot be less than one square inch, nor the size of rod be less than 1/2 inch. Therefor there must be 4 rods 5/8" round; area = 4 x .3 = 1.2 sq. inches. For ties use 1/4" round rods spaced 7 1/2" centers which is 12 times least diameter of 5/8" rods.

For the second story column, the load is 94300#; and refering to Table 5, it will be noted a 16"x16" size is required having a steel percentage of p = .028. As the area of the core = 13 x 13 = 169 sq. ins., the amount of steel = .028 x 169 = 4.73 sq. in. = 8 round rods 7/8" diameter; actual area = 8 x .6 = 4.8 sq. in. For ties use 1/4" round rods spaced at 10 1/2" apart which is 12 times the diameter 7/8". The vertical steel should extend upward into the next column far enough to develop bond for the stress in the steel; this may be taken at n times the concrete stress or 15 x 400 = 6000# per sq. in. For deformed rounds with allowable bond stress of 100# per sq. in. 15 diameters would be required or about 13" but use 16"

—STRUCTURAL DESIGN—

so the length of rods will come out even for a story height of 11'8".

For the first story column the load is 166500 lbs. A 21" x 21" size having a core area of 18 x 18 = 324 sq. in. would be suitable. Taking p = .02, the steel area = .02 x 324 = 6.48 sq. in. equivalent to 4 rods 1 1/8" and 4 rods 1" round; total area = 7.14 sq. in.

(Actual $p = \frac{7.14}{324} = .022$)

$$P = A\,c\,(\,1 + p\,(n - 1))$$

$$p = 400 \times 324\ (1 + .308) = 129600 \times 1.308 = 170000\#\ \text{O.K.}$$

The ties 1/4" round should be spaced 12" centers = 12 diameters of smallest rods. Extend the rods up into the next story 15 diameters for bond which amounts to 15 x 1 1/8 = 17" say 19". Then the length of vertical steel is 11'8" plus 1'7" = 13'3".

For the basement story column the load is 236900#. A size 24" x 24" with core area of 21 x 21 = 441 sq. in. would be suitable. With p = .025; A = .025 x 441 = 11 sq. in. equivalent to 8 rods 1 1/4" square = 8 x 1.56 = 12.5 sq. in.

$p = \frac{12.5}{441} = .0284$ and $P = A\,c\,(1 + p\,(n - 1))$.

$$P = 441 \times 400\ (1 + .398) = 176400 \times 1.398 = 246000\ \#\ \text{O.K.}$$

The ties of 1/4" round rods should be spaced 12 diameters of the smallest vertical steel = 12 x 1 1/4 = 15" centers. For bond extend the vertical 24" into next story.

--STRUCTURAL DESIGN--

COLUMNS. TYPICAL WALL COLS.

The wall columns carry the brickwork, spandrel beams and weight of the columns themselves; the floors are carried by the interior columns, and independent framework for cold storage buildings where the insulation entirely separates the walls from the interior. The size of these columns is predetermined by the available space in the brick walls conforming with the architectural design. The outside pilasters being 30" wide, and the offset courses 4 1/2" each leaves a width of 21" for the column proper; while the thickness is 12 1/2" the same as the brick curtain wall. The loads for a 17 foot panel are:-

3rd Story	13500	13500
2nd "	32400	45900
1st "	32400	78300
Basement Story	32400	109600

These are the full dead loads, figuring 120# per cubic foot for brickwork and 150# per cubic foot for concrete beams and columns.

For the 3rd story column the minimum steel must be used for the fixed size 21 x 12 1/2 = 264 sq. in.; area of core = 18 x 9 1/2 = 171 sq. in. The smallest size rods allowed are 5/8" diameter; four rods have a total area of 4 x .3 = 1.2 sq. in. Then $p = \frac{1.2}{171} = .007$; this is sufficient because p may be as low as .005; but 1/2" rods would make p = .0046 which is too small

-- STRUCTURAL DESIGN --

$P = A_c \ (1 + p \ (n - 1))$

$p = 171 \times 400 \ (1 + .098) = 68400 \times 1.098 = 75000\#$

But the load for the 3rd story column is 13500 and for the 2nd story 45900 consequently this size will do for both.

For the 1st story column (load 78300#) use the same size column but with 3/4" round rods; four rods have a total area of 4 x .44 = 1.76 sq. in. Then $p = \frac{1.76}{171} = .0103$

$P = 68400 \times 1.144 = 78400\#$ O. K.

For the basement story column use 17" x 21" size, projecting inside 4 1/2", with 4 rods 3/4" diameter. Total area is 4 x .44 = 1.76 sq. in. Then $p = \frac{1.76}{252} = .007$

Area of core = 18 x 14 = 252 sq. in.

$P = A_c \ (1 + p \ (n - 1))$

$P = 252 \times 400 \ (1 + .098)$

$P = 100800 \times 1.098 = 109000\#$

This is equal to the load, 109600#, but the column being stiffened by the brick wall possesses added safety.

-- STRUCTURAL DESIGN --

CORNER COLUMNS - EXTERIOR.

The load on the exterior corner columns consists of the brick work, the column itself and the beams framing into it. The columns marked A-1, A-7, E-1, E-7 are tabulated as follows:-

Exterior Corner Column Loads.

Col. Mark	A-1	A-7	E-1	E-7
3rd Story	14600	19600	14600	19600
2nd Story	39100	49600	39100	49600
1st Story	63600	75800	63600	75800
Bsmt. Story	88100	85800	88100	100800

For columns A-1 and E-1, having loads of 88100 lbs. in the basement story, the size is fixed by the pilaster measurements, being made 13 x 21 inches. The core is 3 inches less or 10 x 18 = 180 sq. ins. The smallest percentage of steel that can be used is .5%. Therefore the steel area would be .5% x 180 = .9 sq. ins. But the minimum steel allowed in any column is 1 sq. in. so the smallest rods that could be used would be 4 round rods 5/8" diameter, making a total area of 1.2 sq. ins. The load that such a column would sustain is determined by the formula:-

$P = A\,C\,(1 + p\,(n - 1))$

$P = 180 \times 400\,(1 + .0067\,(15 - 1))$ since

$p = 1.2/180 = .0067$, then

$P = 72000 \times 1.094 = 79000$ lbs.

-- STRUCTURAL DESIGN --

This column will do for the first, second and third stories but not for the basement. Therefore using the same size 13 x 21, try 4 round rods 1" diameter. The steel area is 4 x .7854 = 3.14 sq. ins., and p = 3.14/180 = .0175. Using the same formula as above, the load it will sustain is,

$$P = 72000 \times 1.244 = 89500 \text{ lbs.}$$

which is in excess of the maximum load in the basement story of 88100 lbs. This column will be the same size and shape in all stories, except that 1" rods will be used in the basement column and 5/8" rods in the columns above.

The size of column A-7 is determined by the pilaster measurements. It is 21" x 21" for the basement and first stories; for the second and third stories it is ell-shaped, 21" each way. With p = .005, the steel area is .005 x 324 = 1.62 sq. ins. since the core is 18 x 18. The maximum load it will sustain is (Using 4 round rods 3/4"):-

$$P = 324 \times 400 \times 1.076 = 140000 \text{ lbs.}$$

Although this is in excess of the actual load 85800 yet it is the least that can be used and will apply to the basement and first stories. For the second and third stories the core is 282 sq. ins. With p = .005, the steel area is 1.41 sq. ins. so that 4 round rods 3/4" would have to be used the same as for the lower columns, consequently the columns would be ample from top to bottom.

-- STRUCTURAL DESIGN --

For column E-7 the size in the basement is also 21 x 21 and with 4 round 3/4" rods the load it would carry amounts to 140000 the same as A-7. But the greatest load is 100800 lbs. so column E-7 would be patterned after A-7. See column schedule for sizes of columns in each story, also for diameter and length of rods.

In the case of the ell-shaped columns where 4 rods do not work well in arrangement 8 smaller rods are substituted as indicated on the column schedule.

-- STRUCTURAL DESIGN --

COLUMNS - INTERIOR WALL COLUMNS.

Since the floors are supported independently of the walls, columns must be provided inside the cork board layer to support the edges of the floors in the end bays. The width of these columns is determined by the outer columns 21" and the thickness by the height of not less than 1/16 of 11' 8" = 9". The core area = 6 x 18 = 108 sq. in. Minimum steel = .005 x 108 = .54 sq. in. but not less than 1 sq. in. can be used so the least is 4 round rods 5/8" diameter = 1.2 sq. in. This column will support a load $P = Ac(1 + p(n - 1))$. $p = \frac{1.2}{108} = .011$

$P = 108 \times 400 (1 + .155) = 67000$ lbs.

The floor loads on these columns for the various stories are as follows;

Story	Increment	Load
3rd	18000	18000
2nd	32500	50500
1st	37200	87700
Bsmt.	37300	125000

Consequently the minimum column may be used for the 2nd and 3rd stories the load being less than 67000 lbs.

For the first story column, a size 21 x 12 must be used; size of core = 18 x 9 = 162

If p = .0245 then area of steel = .0245 x 162 = 3.96 sq. in. = 4 round rods 1 1/8" diameter. The load $P = 162 \times 400 (1 + .134) =$

-- STRUCTURAL DESIGN --

87700 # O.K.

For the basement column use a size 21 x 16; core = 18 x 13 = 234 sq. in. Try 6 round rods 1 1/8" diameter. Then area of steel = 6 sq. in. and $p = \frac{6}{234} = .0256$

$P = 234 \times 400\ (1 + .36) = 127000$# This is enough since the actual load is 125000#.

For the interior wall columns at the corners, the size is 12 x 12" with a core equal to 9 x 9 = 81 sq. in. The loads are as follows;

Story	Increment	Load
3d	10000	10000
2nd	17000	27000
1st	195000	46500
Bsmt.	19500	66000

With minimum steel of 4 round rods 5/8" diameter with area of 1.2 sq. in., $p = \frac{1.2}{81} = .0148$ and the load the column will support is:-

$P = 81 \times 400\ (1 + .21) = 39000$# consequently this will be the size used for the 3d and 2nd stories. But for the 1st and basement stories where the loads are 46500# and 66000# respectively, the size of the column used will be larger because the maximum steel of $p = .03$ will not give a column sufficiently large.

For the 1st. story use a 14" x 14" size with a 11" x 11" core = 121 sq. in. Four round rods 1" = 3.14 sq. in. and $p = \frac{3.14}{121} = .026$ $P = 121 \times 400\ (1 + .36 = 66000$# This is the right size for the basement and first stories.

-- STRUCTURAL DESIGN -*

FOOTINGS FOR COLUMNS.

The soil as shown by excavations consists of a top layer 5 feet thick of loam and gravel mixed; below that for a further depth of 25 feet a layer of pure clay containing some gravel in the upper portion. This bed will wupport the building since the basement footings will be about 10 feet below grade. According to the Chicago Code the allowable soil pressure is 3500 lbs per square foot.

For the interior columns the footings will be reinforced concrete of the flat spread foundation type and square in shape. Following the theory developed by experiments at the University of Illinois, the projecting portions of the footings will be considered as cantilevers with a cross section at the edge of the column equal to the width of the column plus a distance on each side equal to the depth of the footing; while the height of the cross section is the same as the depth of the footing. The pressure of soil is exerted over the area bounded by the edge of the column, the outer edge of the footing, and the two diagonal lines running from the corners of the column to the corners of the footing. The center of pressure for this trapezoidal figure is taken at .6 the distance from the edge of column to edge of footing, measuring out from the column face.

For shear the same section area is taken for resisting shear as for figuring moments; and the value of the shear equals the total

pressure on the trapezoidal figure.

Consider the typical interior column. Its load at the foot is 236900 lbs. as given on the page of column design. At 3500 lbs per sq. ft. the area of footing required is 236900 ÷ 3500 = 67.5 sq ft. The nearest size of a square of this footing would be 8' 2". The column measures 24" x 24", and the footing therefore projects 3' 1" beyond the faces of the folumn. Assume a depth of 24" for the footing. Then one edge of the propezoid measures 8' 2" and the other equals 24" + (2 x 24") = 6'0". The area = (8' 2" + 2' 0") x $\frac{3'\,1''}{2}$ = 20.3 sq. ft. Total pressure = 3500 x 20.3 = 71000#. Bending moment = 71000 x .6 x 3.08 x 12 = 1,570,000 in lbs. Taking d = 21", the steel area required to resist this moment = $\frac{1570000}{.87 \times 21 \times 18000}$ = 4.8 sq. ins. But this is distributed over a width of 6' 0" so that the steel area per foot is $\frac{4.8}{6}$ = .8 sq. ins. = 1/2" rounds @ 3" centers.

The shear = 71000 lbs. at the edge of the column but the dangerour section is at a distance from the column equal to the depth d or 21 inches. Here the shear is 36000 lbs and the unit shear = $\frac{36000}{72 \times 21 \times .87}$ = 86# per sq. in. The bond stress = $\frac{71000}{7.13 \times 4 \times 1.57 \times .87 \times 21}$

Therefore hook the ends of the bars. Now test for punching shear using an allowable stress of 120 lbs. per sq. in. and taking the perimeter of the column multiplied by the depth of footing for the area subjected to punching shear.

$$\text{Punch. shear} = \frac{236900 \text{ (total load)}}{24 \times 4 \times 24} = 103\# \text{ per sq. in.}$$

which is less than the allowable and therefore safe.

-- STRUCTURAL DESIGN **

Rods will run in both directions across the footing. The minimum spacing of 3" will affect the rods in the middle portion, a width of 6 feet; the rods beyond that may be spaced farther apart or at 6" centers.

The exterior wall column footings are to be figured the same way when standing alone but if adjacent to an inner column, the two footings are to be combined.

-- STRUCTURAL DESIGN --

FOOTINGS FOR COLUMNS ALONG NORTH WALL.

Since there is already a building at present along the North wall of the Cold Storage House the new footings will not only be carried below the old footings but will not be permitted to extend beyond the building line. Consequently spread footings of the beam cantilever type will be used to support the wall columns and the interior columns of the adjacent bay.

This footing must be arranged so that the center of gravity of the two column loads will coincide with the center of gravity of the footing, the latter representing the center of the upward earth pressure or soil reaction.

The loads are:-

On the exterior wall column	109600
" " interior " "	125000
" " interior floor	236900
Total	471500

See Fig. 6 (Diagram)

For a soil pressure of 3500# per sq. foot the area of the footing will be $\frac{471500}{3500} = 135$ sq. ft. Now take moments about the point A to determine the center of gravity of the column loads.

$$x = \frac{(125000 \times 12.75) - (109600 \times 14.25)}{471500} = 6'8''$$

Therefore the center of gravity of the footing must be 6'8" from the point A of 8'4" from the extreme wall end. Making the shape of the footing rectangular puts this point in the middle and the

-- STRUCTURAL DESIGN --

other end of the footing must be 8'4" in the opposite direction. The total length is then 16'8" and it projects 1'8" beyond the center of the interior floor column. Dividing the total area 135 sq. ft. by the length 16'8" gives the width as 8'2".

The soil pressure conforms to a uniform boad and the bending moment near the middle of the footing is 6,900,000 inch-lbs. Let the depth of footing equal 33", then d = 30" allowing 3" of concrete covering. Area of steel per foot width $= A = \frac{6900000}{8.1 \times 15800 \times 30} = 1.8$ sq. ins. This is equivalent to 3/4" round rods at 3" centers in top of footing. Use the same size rods in the bottom of footing but space them 12" centers just to prevent cracks from stresses due to uneveness of soil pressure. Also put in some 1/2" round cross rods at 24" centers to prevent cracks, using these rods as ties for top and bottom layers.

The shear $V = 236900$ lbs. and $v = \frac{236900}{12 \times 8.1 \times 30 \times .877} =$ 93# per sq. inch. Stirrups will be required spaced as follows:- 4 at 8" - 4 at 10" bending the stirrups up twice or "W" shaped so the resistance of four times the cross section will be effective. The long top rods must be bent at the 5th points like beam bars and hooked at the ends for bond stress. Every alternate rod to be bent, the others running straight through

but hooked at both ends.

The other wall column footings may extend beyond the property lines and so are figured in the same manner as the typical interior footings.

The concrete wall between the exterior columns and extending to the first floor level when the brickwork starts will be made 17" thick and will be reinforced horizontally by 5/8" round rods at 12" centers on outside and inside face, and vertically by 5/8" round rods at 9" centers in both faces. This wall will rest on top of the column fottings at the columns. it is shown by light lines on the footing drawing.

All footings are to be provided with 1" round dowels 4'0" long extending 18" into footing and 30" into columns, each footing to have as many dowels as there are column rods and arranged in the same manner so they may be wired to the column reinforcement.

-- STRUCTURAL DESIGN --

INSULATION OF COLD STORAGE WALLS.

In modern refrigeration it is the present commercial practice to employ cork board as an insulating material. The building is completely enveloped in a box of cork by a special type of construction. This consists in building the outside walls independent of the main structure and utilizing a separate system of interior framework to support the floors and their loads. A space is left between the outer walls and the inside framing to accomodate the necessary layers of cork board. The only connection across the space is a series of steel anchors at each floor level and at each column, tying the adjacent columns (exterior and interior) together.

The space between the walls and the inside framework is made 7" wide to allow two layers of 3" corkboard to be placed after the structure is completed. On the roof two layers of 2" corkboard will be used besides the concrete roof slab to allow ample protection from the heat of the sun.

The various floors will be insulated also, using two layers of 2" cork for the purpose of preventing the transmission of heat from one story to the next where a difference of several degrees in temperature must be maintained.

In the first story it is necessary to insulate the columns also, extending the 3" corkboard around the column from floor to ceiling. This is to prevent frost from traveling through to the basement story by means of the concrete columns which extend through the floor.

-- CORK INSULATION --

To prevent the passage of heat and moisture through the walls corkboard has been found by experience to be an ideal insulation. The heat conductivity of Crescent Corkboard, manufactured by the United Cork Companies, is 6.4 B.T.U. per square foot for one degree difference in temperature per 24 hours, as shown by extended tests.

For a 13" brick wall the conductivity is 7.926 B.T.U. and for 6" thickness of concrete floor it is 17.2 B.T.U. per 24 hours.

As an example of determining the value of insulation with respect to refrigeration, consider the beef freezer room on the first floor where the temperature is to be kept at zero (Fahr.). The mean yearly outside temperature may be taken at 52 degrees for this latitude, then the difference is 52 - 0 = ...52

and the area exposed is, for the outside
wall 30 x 11 = ... 330

The heat transmission per square foot is
for a brick wall .... .956
including 6" cork and cement plaster.

For a difference of 52 degrees this amounts
to 52 x .956 = ... 49.8

and for an area of 330 square feet the total
B.T.S. is 330 x 49.8 = ... 16420
for a period of 24 hours.

-- CORK INSULATION --

In the same way find the heat transmitted through the ceiling, floors and partitions, taking the sum for determining the amount of refrigeration needed in tons per 24 hours, as follows:-

For the ceiling, consisting of 6" concrete, 4" cork board and 1½" cement plaster the conductivity per square foot per degree for a period of 24 hours is	...... 1.452 B.T.U.
Temperature difference between cooler above at 30 degrees and the freezer 30 degrees	...... 30.0
Ceiling area 30 x 34 feet	...... 1020.0
Product of these three factors	 44500
B.T.U. per 24 hours.	
For the walls next to the cork partitions the difference in temperature between the freezer at zero and coolers at 12 degrees is	...... 12.0
The area of the partitions forming two sides of the room is 64 x 11	...... 704.0
The conductivity for 4" cork board and 1½" cement plaster per sq. ft. per degree for 24 hours is	...... 1.68
Product of the three factors	 14200
B.T.U. per 24 hours.	

-- CORK INSULATION --

For the floor the area is 1020

The difference in temperature between the basement at 55 degrees and the cooler at zero is 55

The coefficient of conductivity for the floor consisting of 6" concrete, 4" cork and 4" concrete wearing surface is 1.4

Product of the three factors (B.T.U.) = ... 78540

For the wall next to the old building the area is 34 x 11 feet 374

The difference in temperature between the freezer at zero and the old building at 65 degrees is 65

The conductivity of the old and new walls consisting of 13" new brickwork, 4" cork board, 1½" cement plaster, and 13" old brickwork is ...... .05

The product of the three factors is 1220

B.T.U. per 24 hours.

The sum of B.T.U.'s for all surfaces of the room amounts to:-

Outside wall	16420
Ceiling	44500
Cork partitions	14200
Floor	78540
Double wall	1220
Total	154880

-- CORK INSULATION --

To express this quantity in terms of refrigeration it is necessary to define the latter. To melt one pound of ice at 32 degrees (Fahr.) 144 B.T.U. of heat are required. One ton of refrigeration represents the cooling effect produced by melting one ton (2000 lbs.) of ice at 32 degrees into water at 32 degrees: or 2000 x 144 B.T.U.'s = 288000 B.T.U.'s.

Then for the freezer room under consideration where 154880 B.T.U.'s would be required per 24 hours, the equivalent amount of refrigeration in tons is 154880 divided by 288000 = .54 tons per 24 hours. This is for one room only when accidental losses are neglected such as opening of cooler doors, the presence of men in the cooler room and the warming effect of electric lights.

COST INVESTIGATION.

In compiling an estimate of the cost, the building itself will be considered, including excavation, materials of construction, cork board insulation and labor; but not equipment nor machinery for the refrigeration plant. The figures will therefore cover the erection of the cold storage house ready for all ice making installations.

The subject divides itself into four main divisions: excavating, construction, insulation and engineering.

EXCAVATING.

The tract of land being fairly level, the depth which must be dug may be taken at an average of 11'0" from grade to lowest point of foundation. Allowing an additional two feet excess on all sides the area of the hole equals (68 + 4) x (90 + 4) = 72 x 94 = 6768 sq. ft. Multiplying this by the depth 11 feet makes the number of cubic feet 74448; divide by 27 reducing it to 2757.3 cu. yds.

The current price quoted by contractors on work in Chicago is $2.25 per cubic yard making the amount for excavating about $6200.00

CONSTRUCTING.

In analyzing the cost of the concrete work it is necessary to compute the volume of concrete used in floors,

COST INVESTIGATION.

roof, columns and footings. The slab thicknesses for the various stories are :- first, 6½" plus 3" wearing surface = 9½"; second, 6½"; third, 6"; roof, 4½" After deducting the area of cork slots and taking into account the stairs, elevator shaft and columns the area per floor is 5287 sq. ft. Taking the total thickness of all slabs at 30" (Bsmt. 3½") gives the cubic contents of floors at 13217 cu ft.

Cubic contents of columns from base to roof = 3360 cu. ft.

Footings under all columns 4480 cu ft.

Beams extending below slabs 5400 cu ft.

Basement walls 17" thick 6600.

Total 33067 cu. ft. or in cubic yards about 1220.

The steel, figuring p at an average value of .01 amounts to 330 cu ft. and at 480 lbs. per cubic foot equals 160000 lbs or 80 tons.

The concrete at $36.00 per cubic yard totals, including forms and all labor $44000.00

The steel at 16 cents a pound put in place 20300.00

Brickwork in curtain walls amounts to very nearly 3900 cu ft. = 90000 bricks. At present prices for material and labor would amount to $42.00 per thousand or a total of 3800.00

30 cooler doors installed complete @ $65 .... 2000.00

Roofing put on; Carpentry labor and materials; plumbing ducts; plastering; and iron work as determined by estimates used by contractors approximate 16000.00

COST INVESTIGATION.

INSULATING.

Along the outside walls, under the first floor and over the roof slab 6" of cork is used making a total of 21354 sq. ft. of 6" thick cork board. For the partitions, columns and under the second and third floors 4" cork is used making a total of 17174 sq. st. of 4" thickness cork board. Reducing this to cubic feet of cork the amount is 16400 cu. ft. This can be bought and put in at present prices for 1.25 per cubic foot totaling $20500.00

ENGINEERING.

This will include surveying, making drawings and supervising which all together will add 12% to the original estimated cost or 13000.00

SUMMARY OF ALL ITEMS GIVEN ABOVE: -

Excavating	62000.00
Concrete	44000.00
Steel	20300.00
Brickwork	3800.00
Cooler doors	2000.00
Insulation	20500.00
Carpentry & Miscellaneous	16000.00
Engineering	13000.00
ESTIMATE TOTAL	126000.00

COST INVESTIGATION.

Now the cubic contents of the building is 90 x 70 x 55 = 346000 cu ft. and according to recent tabulated costs of actually built cold storage buildings as recorded by the United Cork Co. the complete cost averages about 33 cents per cu. ft. at present prices. This would amount to $114000. without elevators or machinery equipment. In view of present uncertain factors affecting costs, a safe estimate for this building would be around $130,000.00.

LEGEND OF SYMBOLS
AND
ALLOWABLE UNIT STRESSES.

STRESSES IN CONCRETE.

Bending, compression, $f = 700$ #/ sq. in.

Direct compression, $c = 400$ " "

Shear, diagonal tension $v = 40$ " "

Bond, plain round bars $u = 50$ " "

Bond, deformed round bars $w = 100$ " "

STRESSES IN STEEL

Steel, high Carbon, tension, $s = 18000$ #/ sq. in.

Shear, when used as stirrups $y = 12000$ " "

OTHER SYMBOLS.

V = Total shear in beam.

A = Steel area of rods.

p = Steel ratio

b = width of beam

d = depth to center of steel

h = total depth

n = ratio of moduli of elasticity of steel and concrete

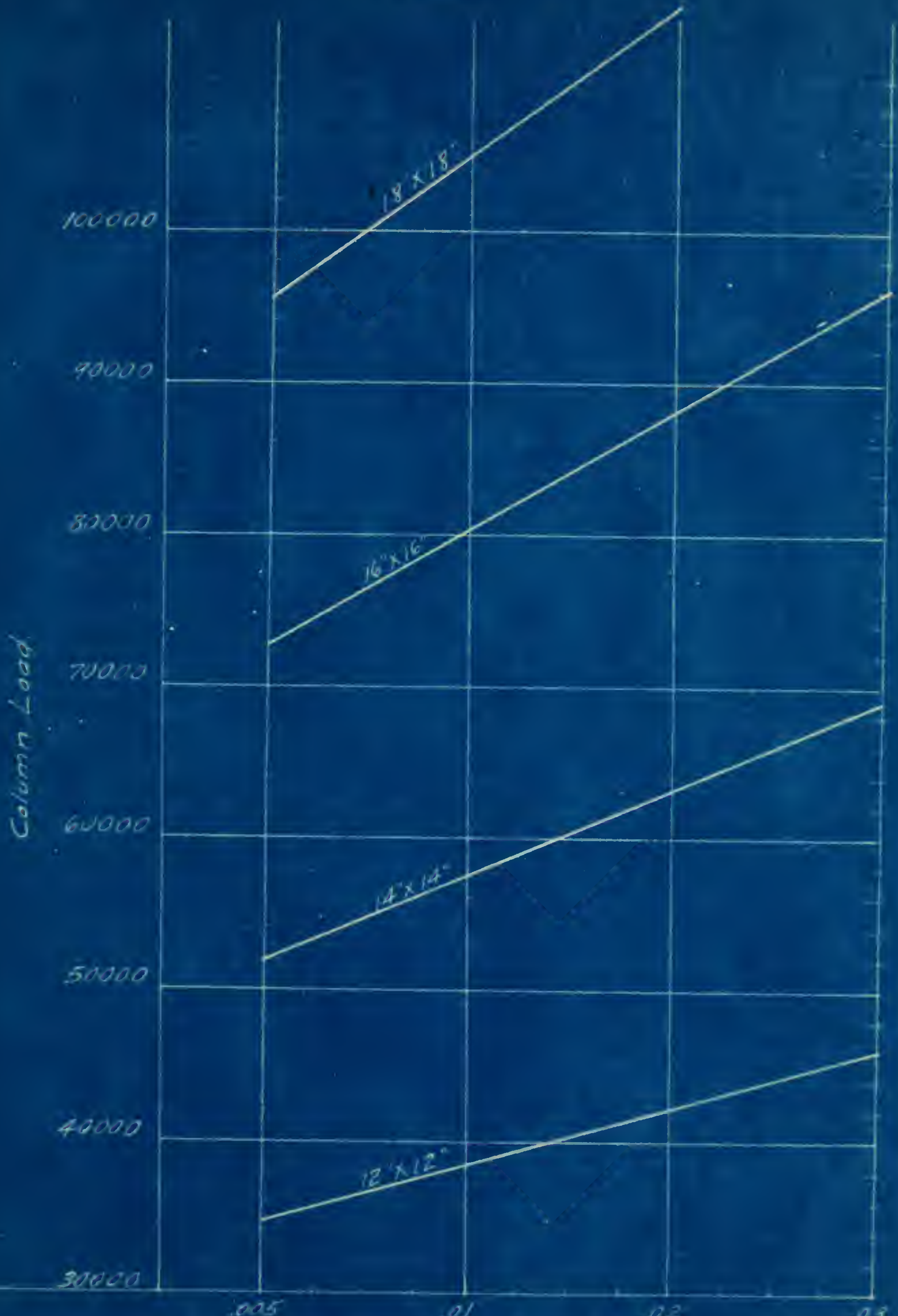
k = ratio of distance down to neutral axis to d.

$j = 1 - \frac{k}{3}$

M = Bending moment

$R = \frac{M}{b\ d^2}$

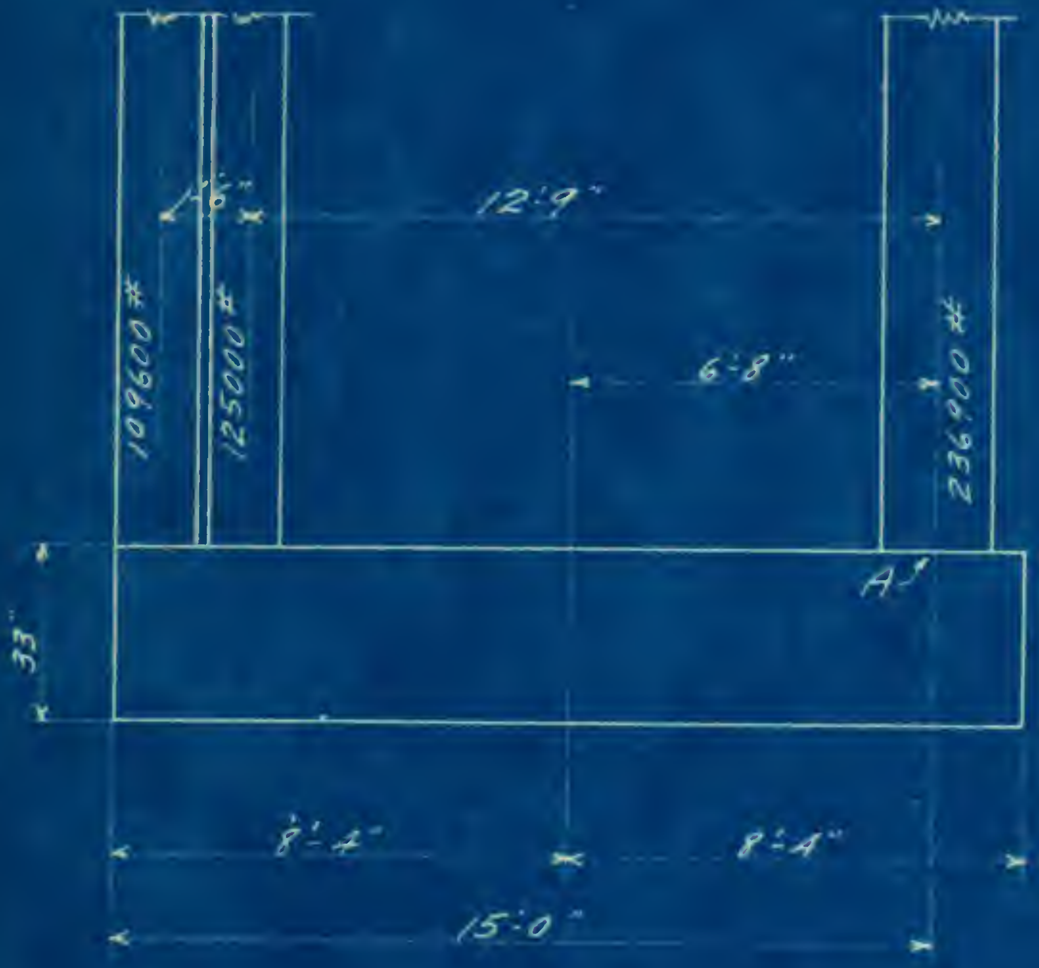
TABLE 5



100000
90000
80000
70000
60000
50000
40000
30000
Column Load
.005
.01
.02
.03
Steel Ratio, p
18"x18"
16"x16"
14"x14"
12"x12"

FIGURE 6.

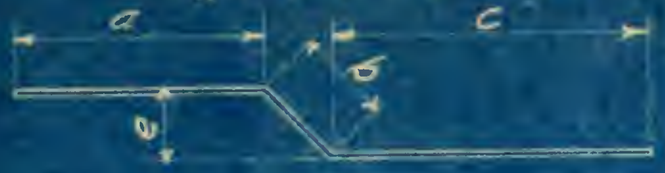
Diagram of spread footing for exterior wall columns and interior floor column with loads as shown. At the left end the edge of footing coincides with building line.



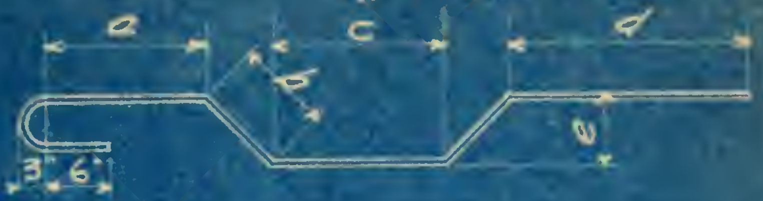
Taking moments about point A gives the distance to center of gravity of the loads as 6'8" and this must coincide with the center of the footing which extends 8'4" each way. See computations for footings of North wall columns.

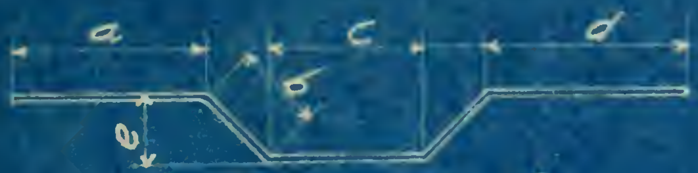
Typical Bending Details - Beam Bars.

Type-aa For slabs

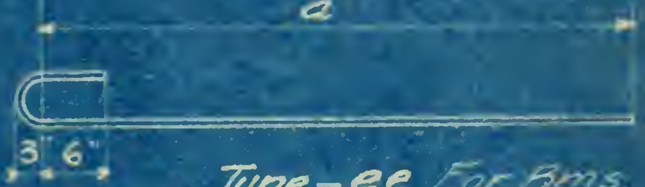


Type-bb For Bms





Type-cc For Bms



Type-ee For Bms.

All dimensions out to out.

All steel plain rounds. - Slabs
" " deformed - Bms

Mark	No.	Size	Length	Type	a	b	c	d	e
2B1	6	5/8φ	21'-9"	bb	2'-1"	20"	8'-0"	7'-1"	14"
2B2	6	3/4"	20'-0"	"	1'-5"	23"	7'-3"	6'-3"	16"
2B3	10	7/8"	20'-0"	"	1'-6"	25"	6'-9"	6'-4"	18"
2B8	4	5/8"	21'-0"	"	2'-1"	20"	7'-6"	6'-10"	14"
2B15	8	3/4"	22'-9"	"	1'-1"	27"	7'-6"	7'-7"	19"
2B16	16	1"	23'-3"	"	1'-3"	38"	7'-6"	6'-11"	27"
2B17	4	7/8"	24'-3"	"	2'-5"	27"	8'-0"	8'-1"	19"
2B18	2	7/8"	24'-6"	"	2'-3"	30"	8'-0"	8'-0"	21"
2B4	12	5/8"	26'-0"	cc	7'-7"	20"	7'-6"	7'-7"	14"
2B5	10	3/4"	26'-0"	"	7'-4"	23"	7'-6"	7'-4"	16"
2B6	20	7/8"	26'-0"	"	7'-3"	25"	7'-6"	7'-3"	18"
2B7	4	7/8"	26'-6"	"	7'-2"	28"	7'-6"	7'-2"	20"
2B24	2	7/8"	29'-0"	"	7'-6"	24"	10'-0"	7'-6"	17"
2B20	6	3/4"	29'-6"	"	8'-3"	27"	8'-6"	8'-3"	19"
2B21	14	1"	30'-0"	"	7'-7"	38"	8'-6"	7'-7"	27"
2B19	4	5/8"	29'-6"	"	8'-6"	24"	8'-6"	8'-6"	17"
2B22	6	7/8"	29'-6"	"	8'-3"	27"	8'-6"	8'-3"	19"
2B2	6	5/8"	14'-0"	ee	12'-9"				
2B3	10	7/8"	14'-0"	"	12'-9"				
2B8	4	5/8"	15'-0"	"	13'-9"				
2B11	2	7/8"	15'-6"	"	14'-3"				
2B14	4	5/8"	17'-6"	"	16'-3"				
2B16	16	1"	16'-0"	"	14'-9"				
S-1	428	1/2φ	20'-0"	aa	8'-2"	7"	11'-3"		5"
S-2	243	1/2"	22'-6"	"	9'-2½"	6½"	12'-9"		4½"
S-3	125	1/2"	18'-6"	"	7'-7"	7"	10'-6"		5"
S-4	154	1/2"	21'-0"	"	8'-9½	6½"	11'-8"		4½"
S-5	178	1/2"	20'-0"	"	8'-2½"	6½"	11'-3"		4½"
S-6"	100	1/2"	22'-6"	"	9'-3"	6"	12'-9"		4"
S-7"	88	1/2"	18'-6"	"	7'-5½"	6½"	10'-6"		4½"
S-8"	61	1/2"	21'-0"	"	8'-10"	6"	11'-8"		4"

T 29124
q693 W18

Wamsley, G. R.

Design of 3-story reinforced
concrete cold storage bldg.

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q693 29124 W18

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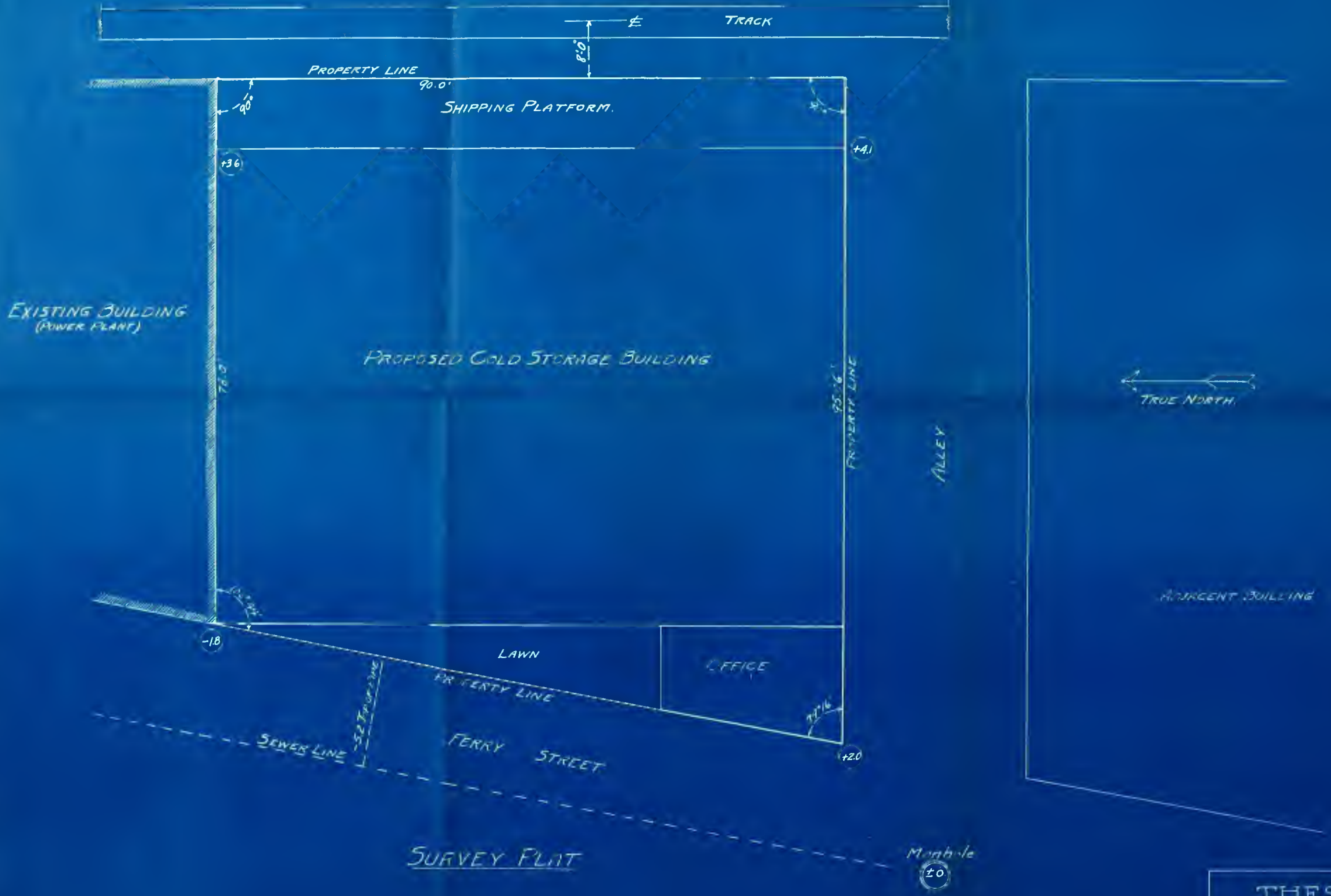
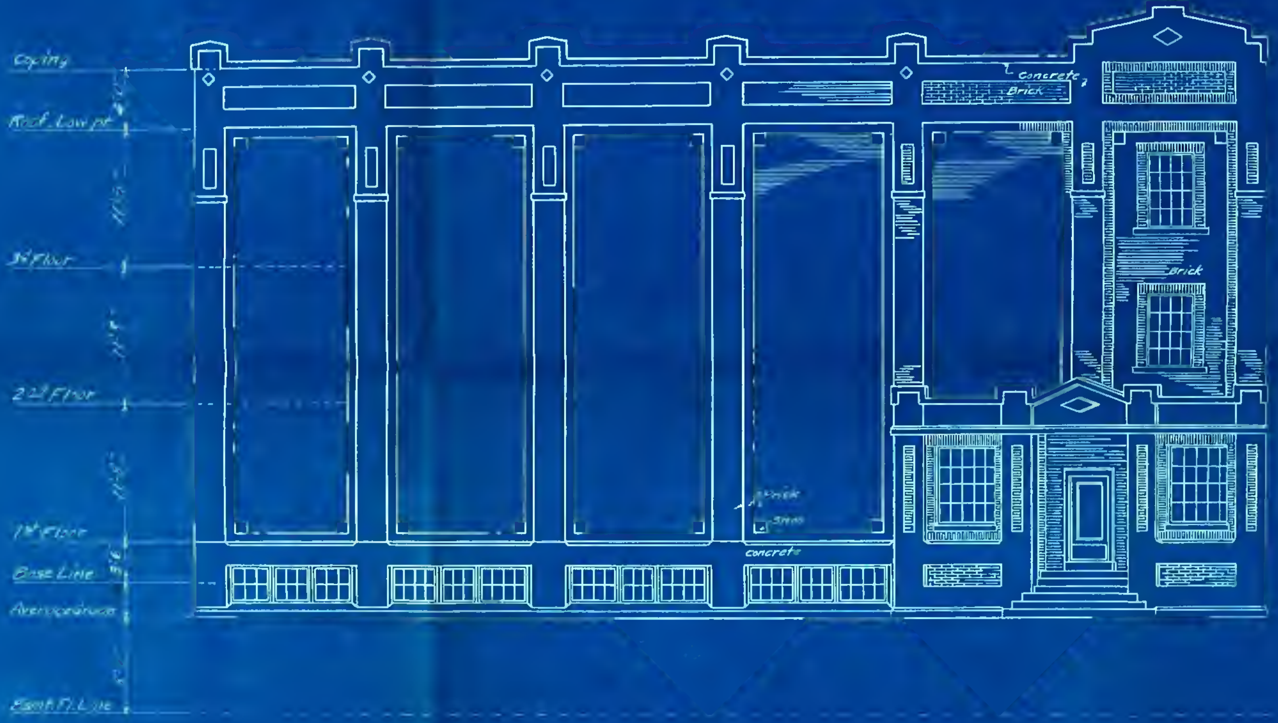


SURVEY PLAT

THESIS
COLD STORAGE BUILDING
SURVEY PLAT

G R Womsley

1



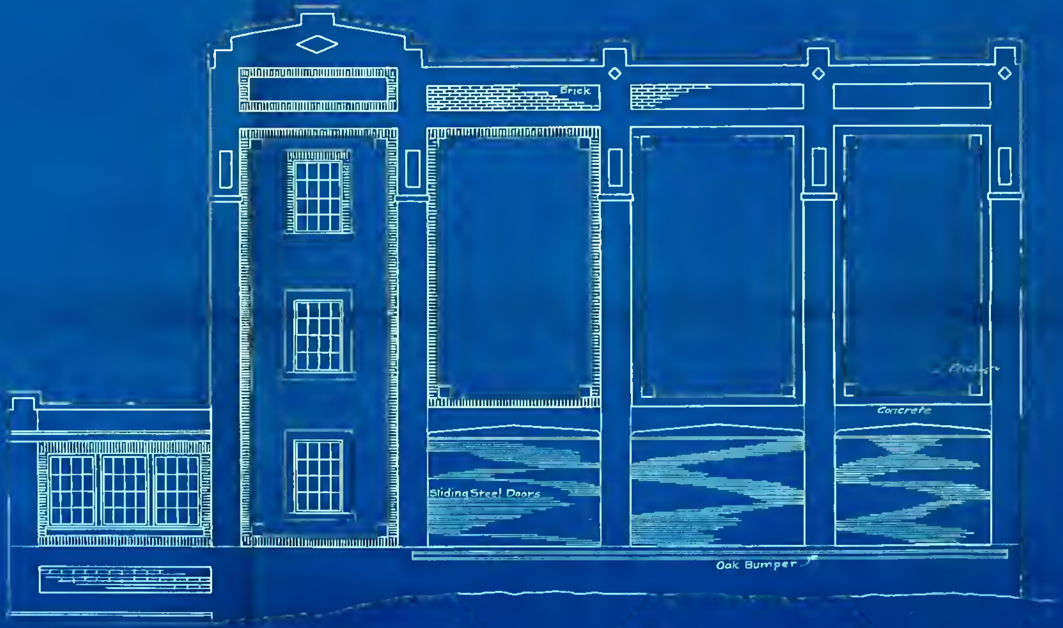
WEST ELEVATION

THESIS

GOLD STORAGE BUILDING
WEST ELEVATION

G. R. Womsley

2



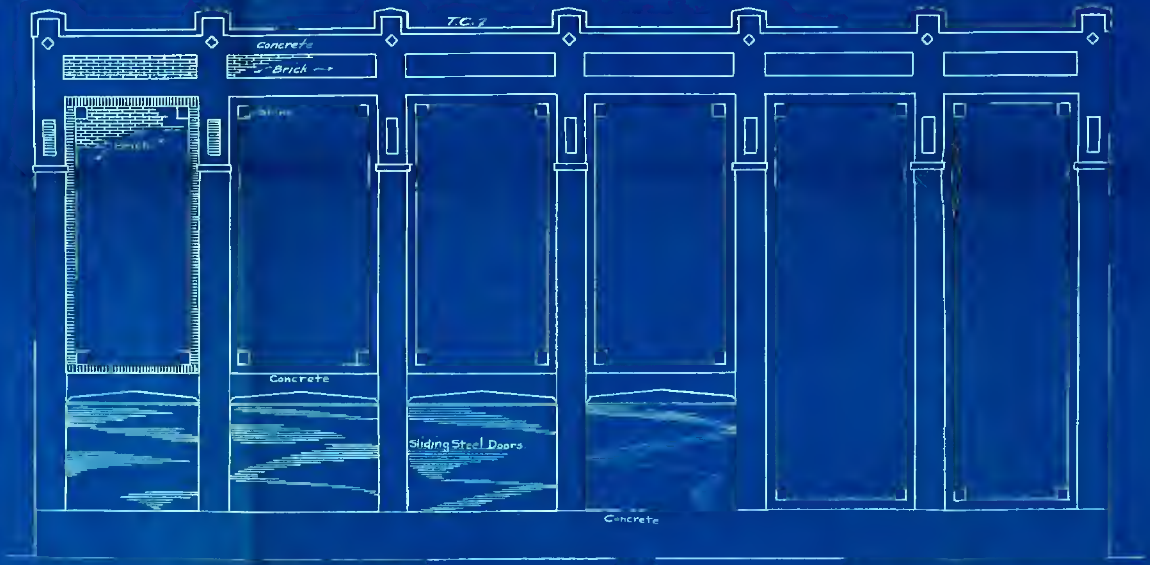
SOUTH ELEVATION.

THESIS

COLD STORAGE BUILDING
SOUTH ELEVATION

G. R. Wamsley

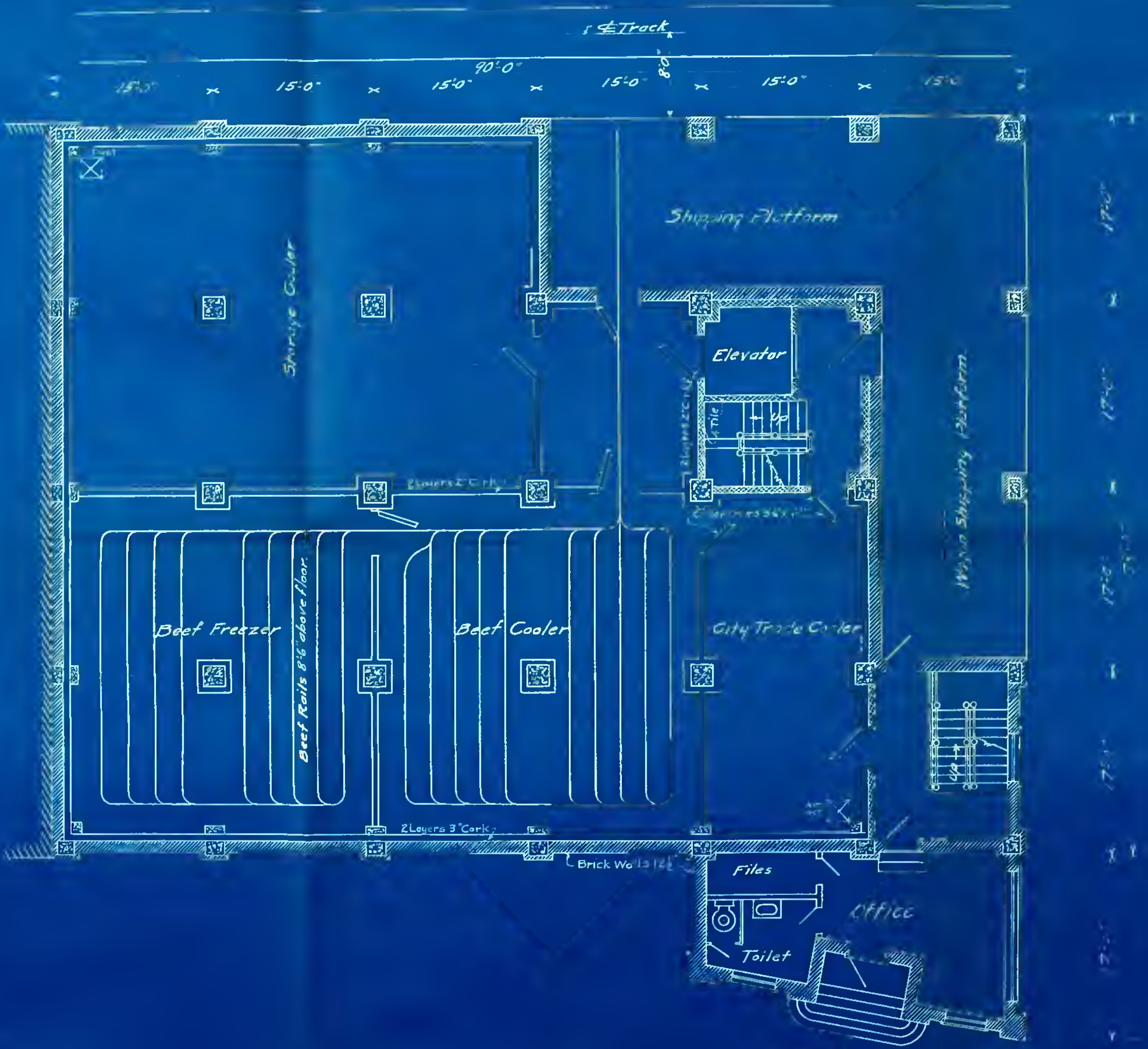
3



EAST ELEVATION

THESIS

COLD STORAGE BUILDING
EAST ELEVATION
G. R. Womsley

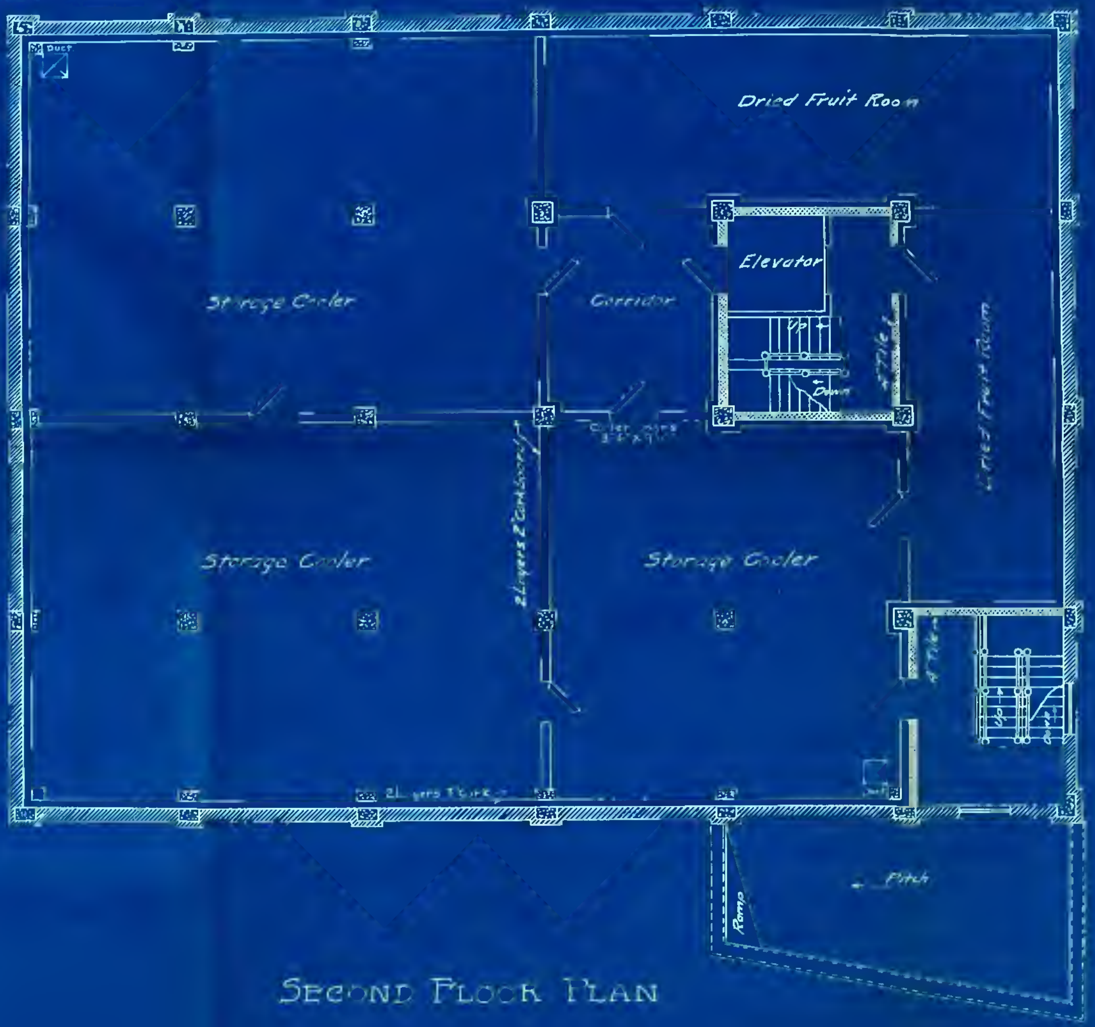


FIRST FLOOR PLAN.

THESIS
COLD STORAGE BUILDING
FIRST FLOOR PLAN

G R Womsley

5



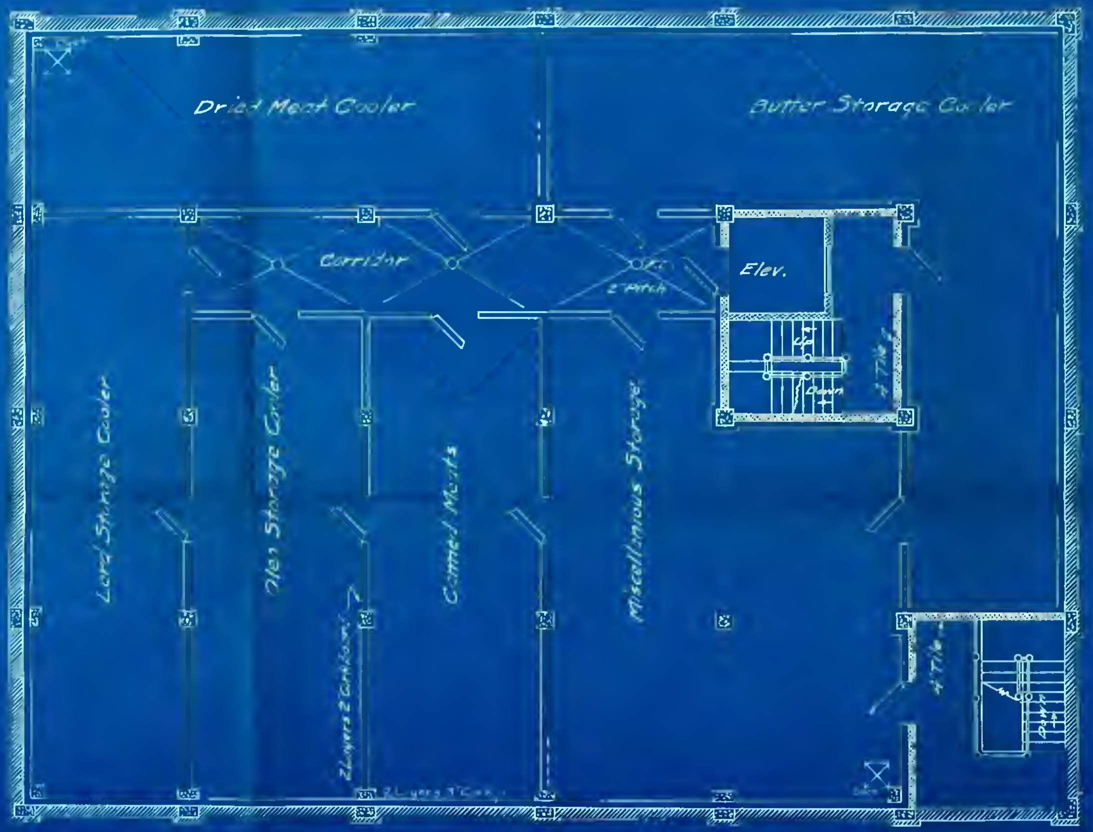
SECOND FLOOR PLAN

THESIS

COLD STORAGE BUIDING
SECOND FLOOR PLAN

G R Wimsley

6

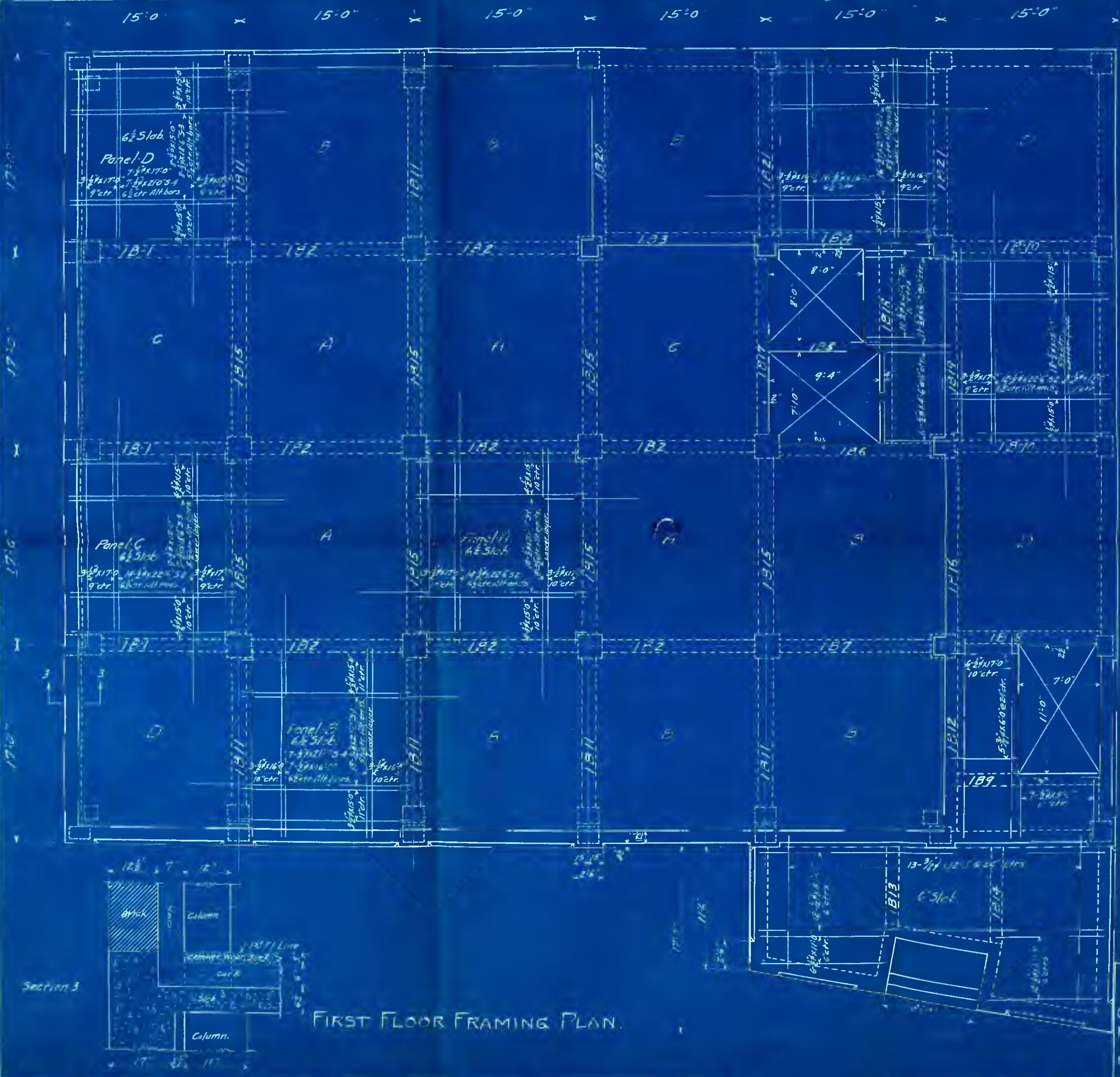


THIRD FLOOR PLAN

THESIS

COLD STORAGE BUILDING
THIRD FLOOR PLAN

G.R. Wamsley



FIRST FLOOR FRAMING PLAN.

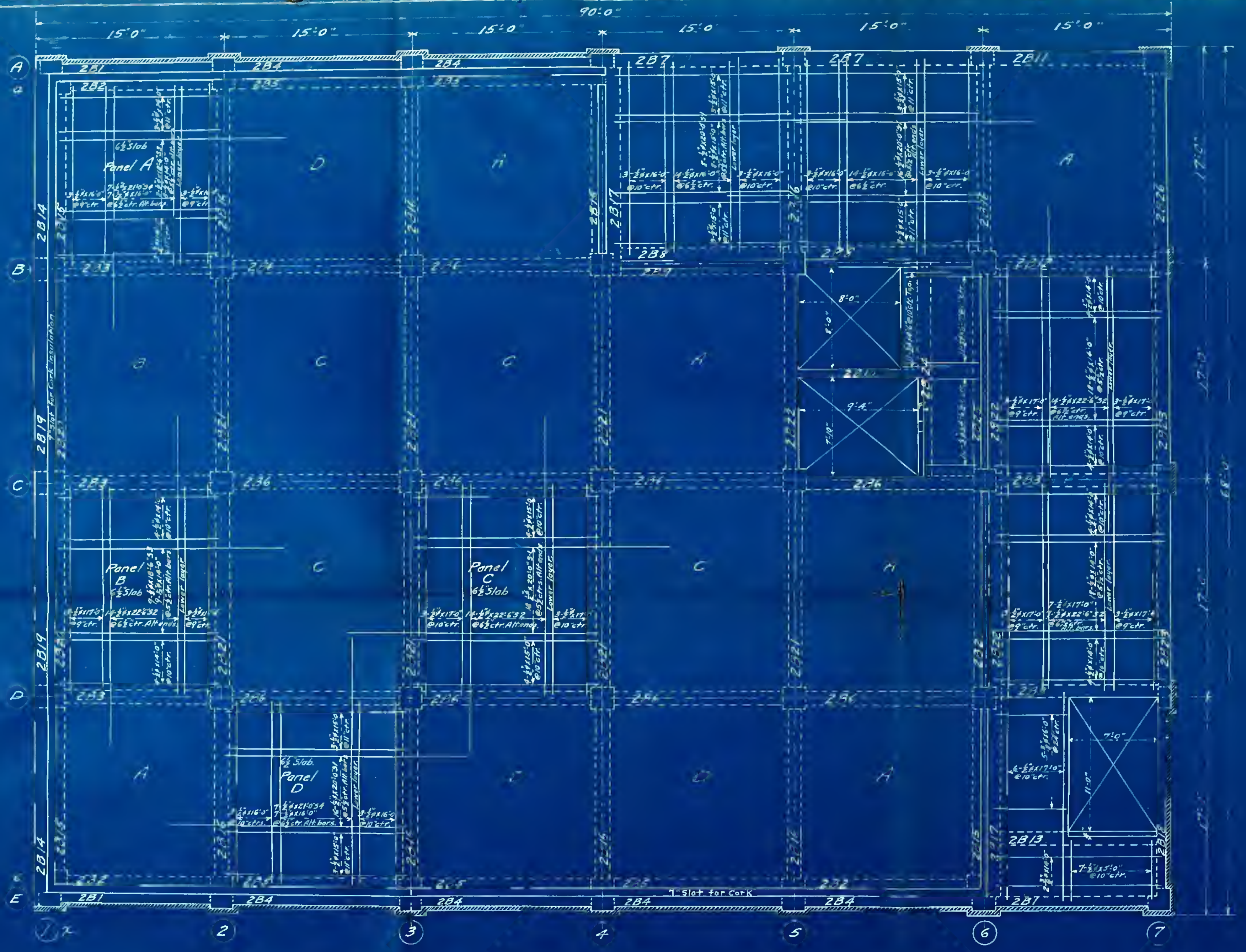
THESIS

COLD STORAGE BUILDING

FIRST FLOOR FRAMING PLAN

G. R. Wamsley.

8



Second Floor Framing Plan

Note For Office Roof Framing see Sheet #8 1st Floor which is framed similar.

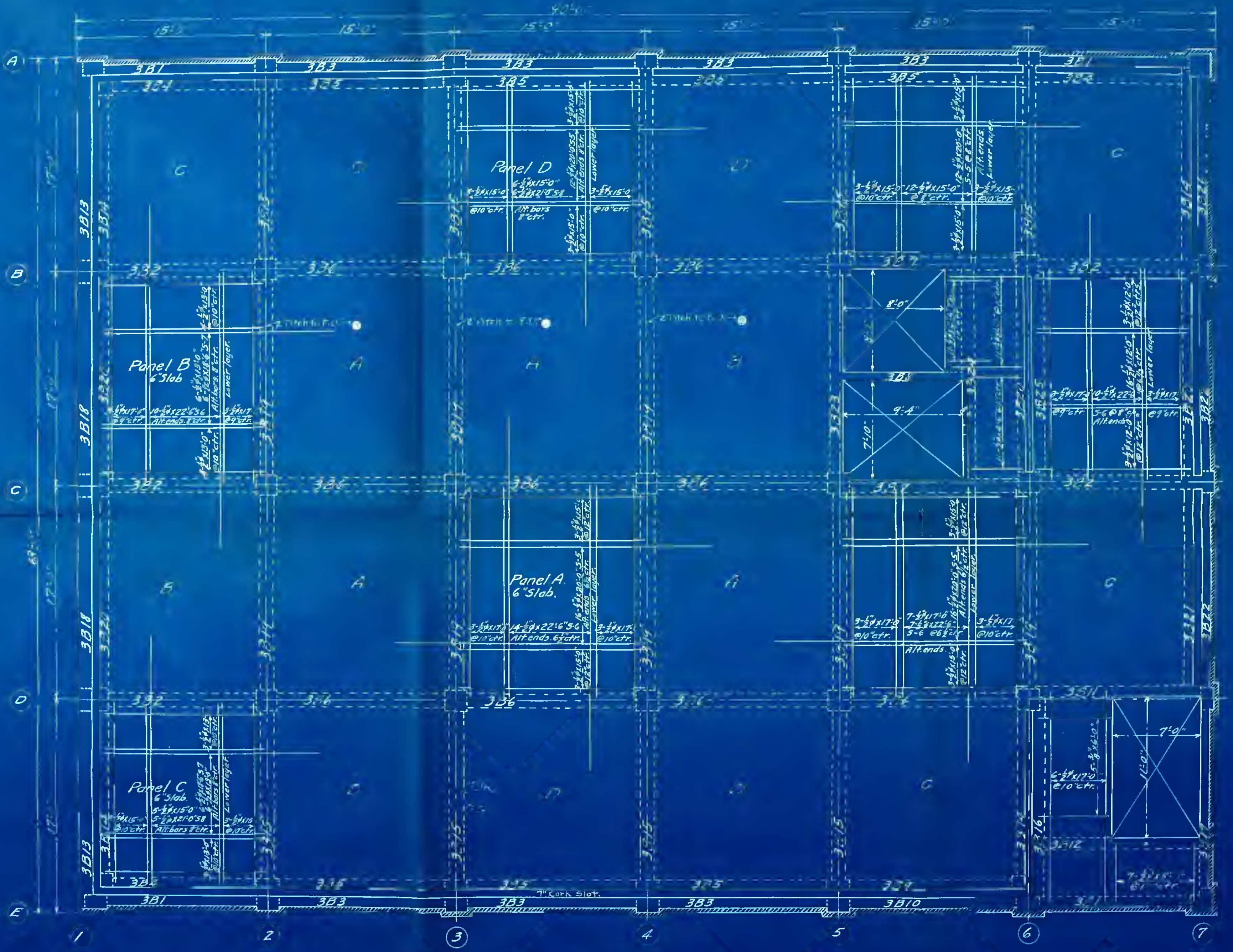
THESIS

COLD STORAGE BUILDING

SECOND FLOOR FRAMING PLAN

G.R. Wantsley

9



Third Floor Framing Plan

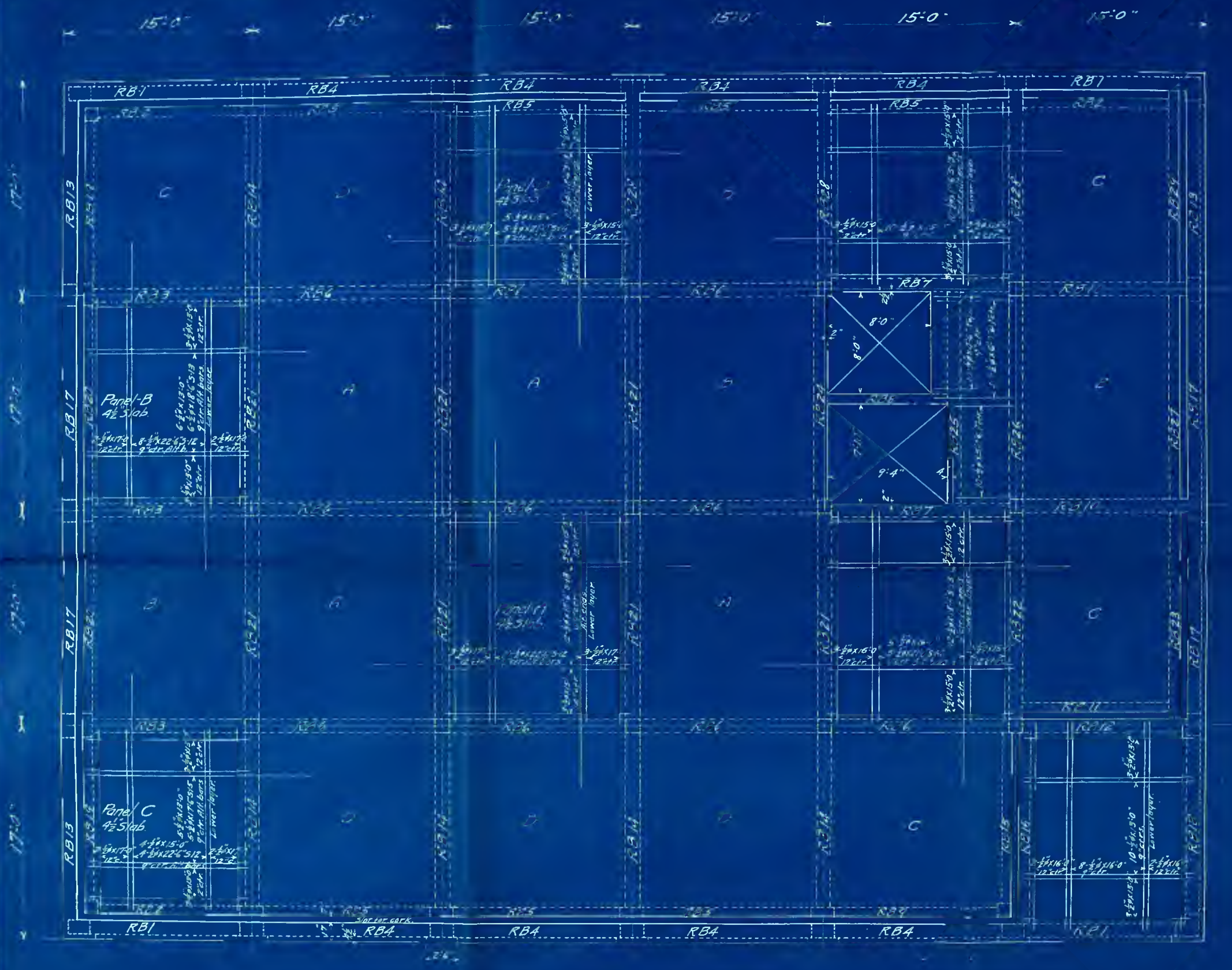
THESIS

COLD STORAGE BUILDING

Third Floor Framing Plan

G.R. Womsley

10

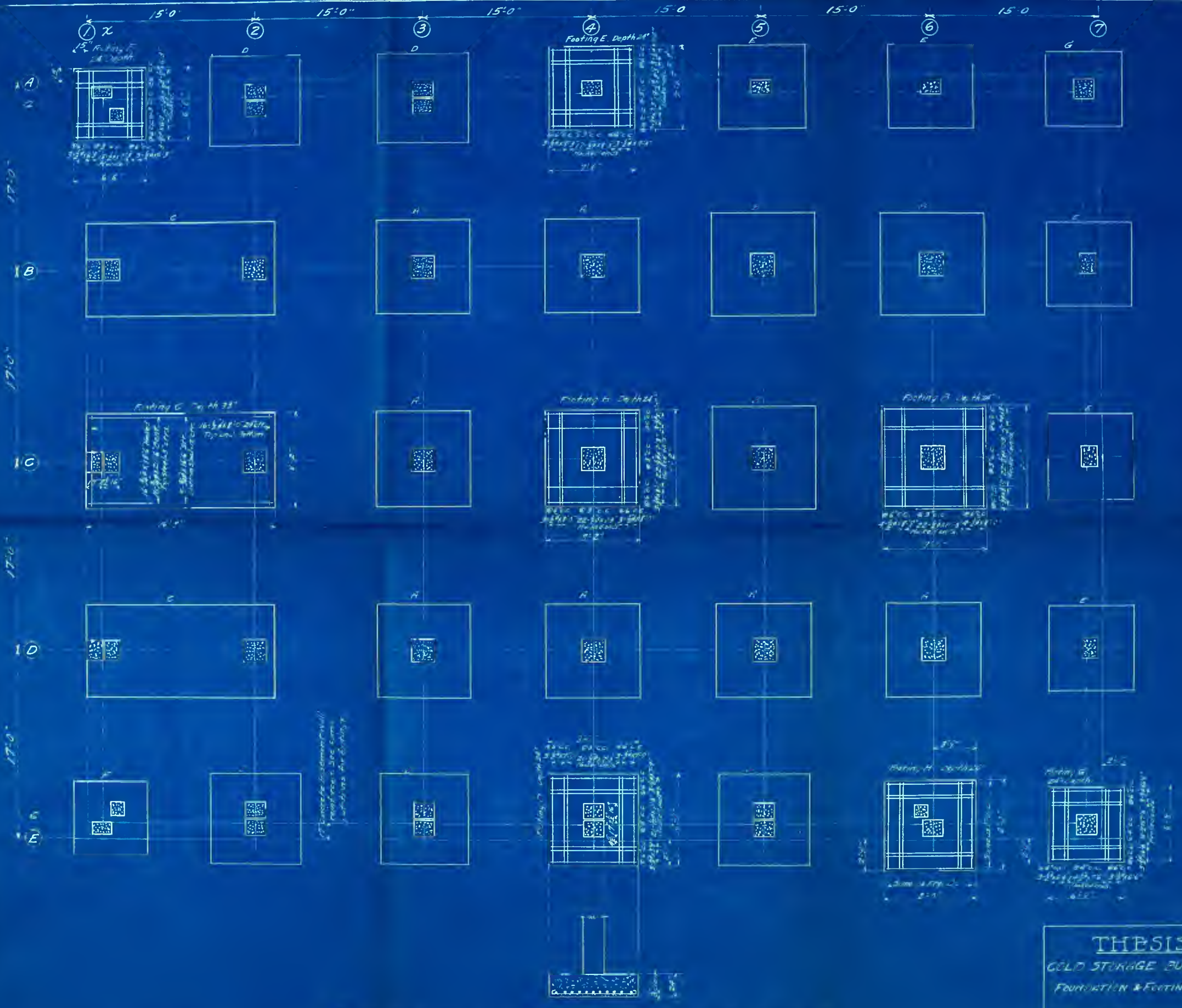


Roof Framing Plan

THESIS
COLD STORAGE BUILDING
ROOF FRAMING PLAN

G. R. Wamsley

11



FOUNDATION AND FOOTING REINFORCEMENT

THESIS

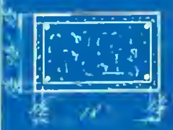
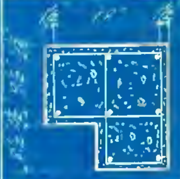
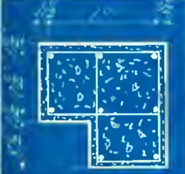
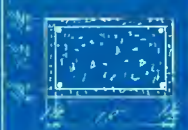
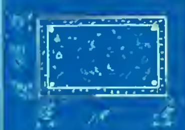
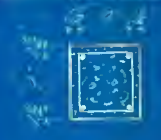
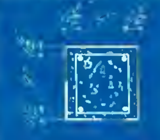
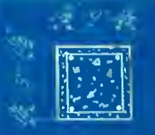
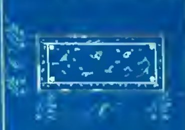
COLD STORAGE BUILDING

FOUNDATION & FOOTING PLAN

G. R. Womsley

12

COLUMN SCHEDULE

	Column Mark	A-1 & E-1	A-7	E-7	A-2-3-4 E-2-3-4-5 B-1, C-1, D-1	A-5-6 E-6 B-7, C-7, D-7	B-5-6 C-5-6	B-2-3-4 C-2-3-4 D-2-3-4-5-6	ax, ex, e6	a-2-3 Bx, Cx, Dx. e-2-3-4-5
	No. of Columns	2	1	1	10	6	4	11	3	9
Third Story.	Sketch									
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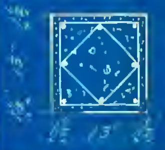
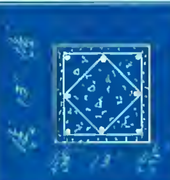
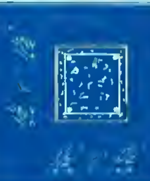
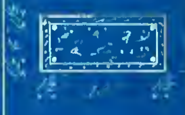
	Ties	19-1/4"φ x 5'-3"	38-1/4"φ x 5'-3"	38-1/4φ x 5'-3"	19-1/4"φ x 5'-3"	19-1/4"φ x 5'-3"	35-1/4"φ x 3'-6"	20-1/4"φ x 3'-9".	18-1/4"φ x 3'-9"	20-1/4"φ x 4'-9"
Second Story	Sketch	See 3d story	See 3d story	See 3d story	See 3d story	See 3d story				
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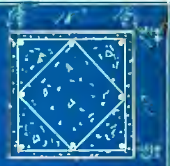
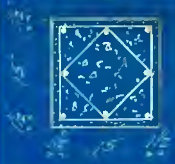
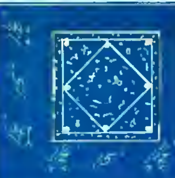
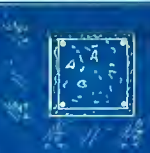
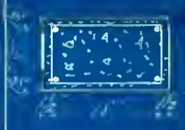
	Ties	18-1/4"φ x 5'-3"	36-1/4"φ x 5'-3"	36-1/4φ x 5'-3"	18-1/4"φ x 5'-3"	16-1/4"φ x 5'-3"	13-1/4"φ x 5'-0 13-1/4"φ x 3'-9"	13-1/4"φ x 5'-0" 13-1/4"φ x 3'-9"	18-1/4"φ x 3'-9"	20-1/4"φ x 4'-9"
First Story	Sketch	See 3d story		See 3d Story	See 3d story	See 3d story				
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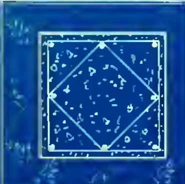
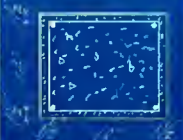
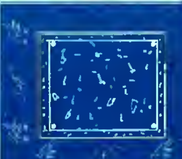
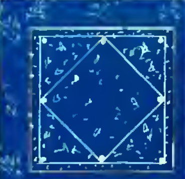
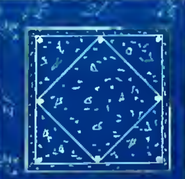
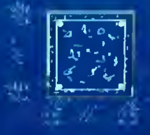
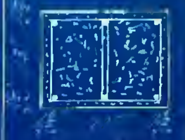
	Ties	18-1/4"φ x 5'-3"	18-1/4"φ x 6'-9" 18-1/4"φ x 5'-0"	36-1/4φ x 5'-3"	16-1/4"φ x 5'-3"	14-1/4"φ x 5'-3"	12-1/4"φ x 5'-9" 12-1/4φ x 4'-3"	12-1/4"φ x 5'-9" 12-1/4"φ x 4'-3"	12-1/4"φ x 4'-3"	12-1/4"φ x 5'-3"
Basement Story	Sketch	See 3d story	See 1st story							
	Verticals	4-1"φ x 14'-0"	8-3/4"φ x 14'-0"	8-5/8φ x 14'-0"	4-3/4"φ x 14'-0"	4-1"φ x 14'-0"	8-1 1/4"φ x 14'-0"	8-1 1/4"φ x 14'-0"	4-1"φ x 14'-6"	6-1 1/8"φ x 14'-6"
	Ties	16-1/4"φ x 5'-3"	16-1/4"φ x 6'-9" 16-1/4"φ x 5'-0"	16-1/4"φ x 6'-9" 16-1/4"φ x 5'-0"	16-1/4"φ x 6'-0"	16-1/4"φ x 6'-0"	16-1/4"φ x 7'-9" 16-1/4"φ x 5'-9"	16-1/4"φ x 7'-9" 16-1/4φ x 5'-9"	16-1/4φ x 4'-3"	16-1/4φ x 4'-6" 16-1/4φ x 4'-6"

Note - Mix 1-2-4 for all columns.

THESIS

COLD STORAGE BUILDING
COLUMN SCHEDULE

G. R. Womsley

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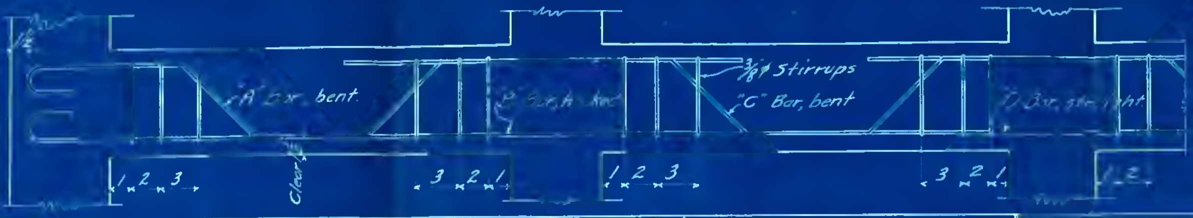
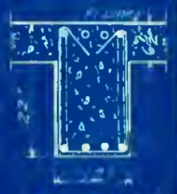
TYPICAL BEAM SCHEDULE.



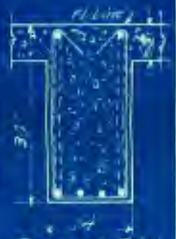
Note.
All beam bars to be one layer only.
All beam bars to be tagged the same as the corresponding beam mark.

Beams				Reinforcing Steel										Stirrups					Remarks
Beam Mark	No. of Bms.	Size		No. Bars per Bm.				Total No. Bars.				Size	Length	No. per Bm.	Total	Size	Length	Spacing in inches from each end	
		W.	D.	A	B	C	D	A	B	C	D								
2B1	3	8"	18"	2	2			6	6			5/8"φ "	21'9" 15'9"	12	36	3/8"	3'9"	3@6 - 3@8"	
2B2	3	9"	20"	2	2			6	6			3/4" 5/8"	20'0 14'0"	12	36	"	4'3"	3@6"- 3@8"	
2B3	5	12"	22"	2	2			10	10			7/8" "	20'0" 14'0"	14	70	"	4'9"	3@6" - 4@8"	
2B4	6	8"	18			2	2			12	12	5/8" "	26'0" 15'0"	18	108	"	3'9"	3@6"-3@8"-3@10"	
2B5	5	9"	20"			2	2			10	10	3/4" "	26-0" 15'0"	18	90	"	4'3"	3@6"-4@8"-2@10"	
2B6	10	12"	22"			2	2			20	20	7/8" "	26'0" 15'0"	18	180	"	4'9"	3@6"-3@8"-3@10"	
2B7	2	12½"	24"			2	2			4	4	7/8" 3/4"	26'6" 15'0"	18	36	"	5'0"	3@6"-5@8"-1@10"	
2B8	2	10"	18	2	2			4	4			5/8" "	21'0" 15'0"	10	20	"	4'0	3@8"-2@10"	
2B9	1	8"	18"	2	2			2	2			5/8" "	21'0" 15'0	10	10	"	3'9"	3@8"-2@10"	
2B10	1	8"	18				3				3	3/4"	11'-6"					Omit stirrups.	
2B11	1	12½"	24	2	2			2	2			7/8" "	21'6" 15'6"	14	14	"	5'0"	3@8"-3@10"-1@12"	
2B12	1	12"	22	2	2			2	2			7/8" "	17'0" 16'6"	16	16	"	4'9"	3@6"-3@8"-2@10"	Both ends of bars to be hooked
2B13	1	12"	22	2	2			2	2			3/4" "	16'9" 15'6"	16	16	"	4'9"	3@6"-3@8"-2@10"	"
2B14	2	12½"	20"	2	2			4	4			3/4" 5/8"	24'0" 17'6"	20	40	"	4'6"	3@6"-4@8"-3@10"	
2B15	4	9"	22			2	2			8	8	3/4" "	22'9" 15'6"	16	64	"	4'9"	3@8"-3@10"-2@12"	
2B16	8	14"	30"	2	2			16	16			1" "	23'3" 16'0"	20	160	"	6'6"	4@6"-3@8"-3@10"	
2B17	2	10"	22"	2	2			4	4			7/8" "	24'3" 18'0"	16	32	"	4'9"	3@6"-5@8"	
2B18	1	8"	24	2	2			2	2			5/8" "	24'6" 17'6"	16	16	"	5'0"	3@6"-5@8"	
2B19	2	12½"	20			2	2			4	4	5/8" "	29'6" 17'0"	20	40	"	4'6"	3@6"-4@8"-3@10"	
2B20	3	9"	22"			2	2			6	6	3/4" "	29'6" 17'0"	16	48	"	4'9"	3@8"-3@10"-2@12"	
2B21	7	14"	30"			2	2			14	14	1" "	30'0" 17'0"	20	140	"	6'6"	4@6"-3@8"-3@12"	
2B22	3	10"	22"			2	2			6	6	7/8" "	29'6" 17'0"	14	42	"	4'9	3@8 - 4@10"	
2B23	2	12½"	24			2	2			4	4	7/8" "	30'0" 17'0"	18	36	"	5'6"	3@6"-4@8"-2@10"	
2B24	1	12"	20"			2	2			2	2	7/8" "	29'0" 18'0"	16	16	"	4'6"	4@6"-4@8"	
2B25	1	8"	16				4				4	3/4"	18'0"	10	10	"	3'6"	5@8"	
2B26	1	12½"	24	2	2			2	2			7/8" "	24'6" 17'6"	18	18	"	5'3"	3@6"-6@8"	

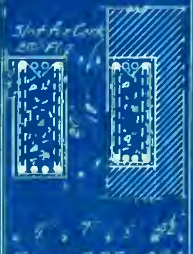
Sketch



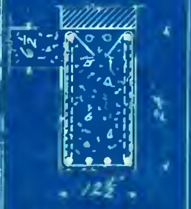
Beam 2B6



Beam 2B21



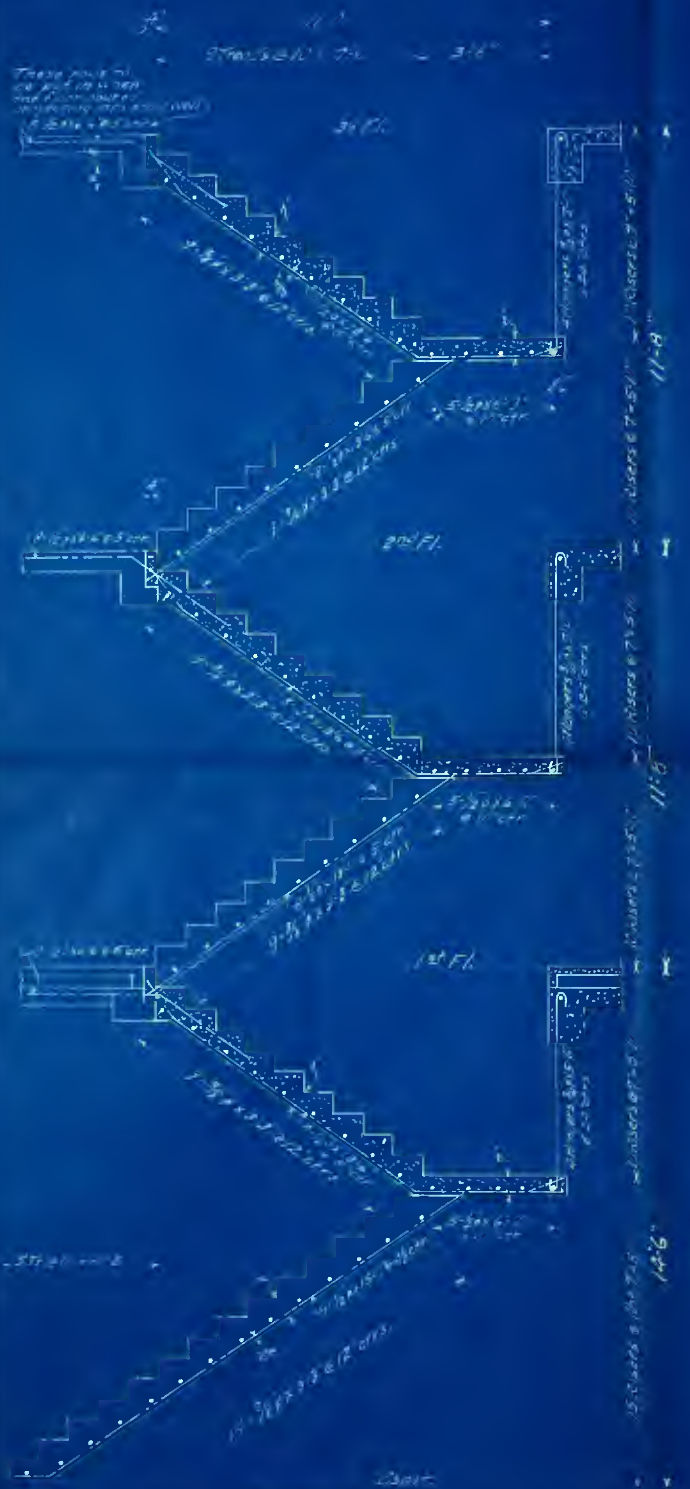
Beams 2B5 & 2B4



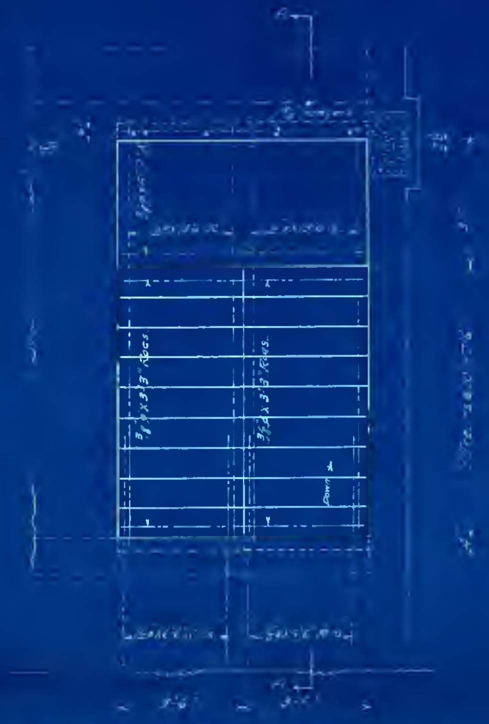
Beam 2B23

THESIS
COLD STORAGE BUILDING
TYPICAL BEAM SCHEDULE

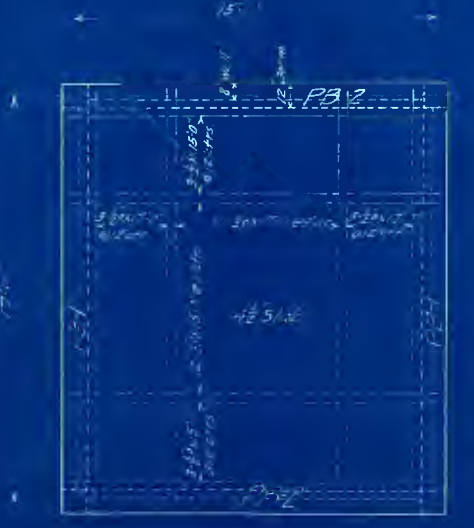
G. R. Wemsley.



Section A-A
Typical detail of reinforcement for Southwest stairs. The S.E. stairs are built similarly.

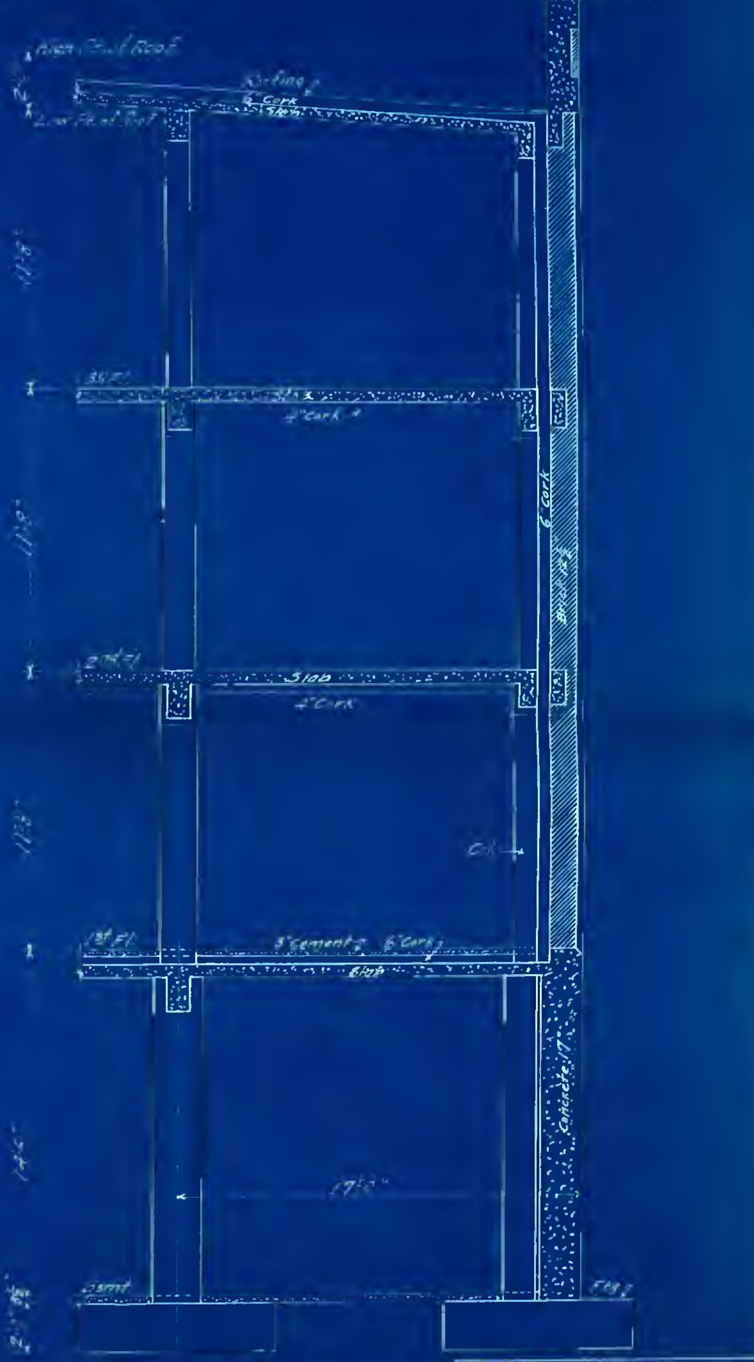


Typical Plan of Stairs at Floor Level



Penthouse Roof Framing

[illegible]



Typical Wall Section
Looking North.

Wall Anchors [illegible] steel plates [illegible] in pairs at each [illegible] on every floor to secure the outer wall [illegible] columns of the inner framing system.

THESIS
COLD STORAGE BUILDING
STAIRS, PENTHOUSE, SECTIONS

G. R. Wamsley

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